A TREATISE

ON THE

Design and Construction

OF

Roofs

BY

N. CLIFFORD RICKER, B.S., M.ARCH., D.ARCH.
Professor of Architecture, University of Illinois; President Illinois Board of Examiners of Architects: Chairman Illinois Commission on Building Laws; Honorary and Active Fellow American Institute of Architects; Fellow American Association for Advancement of Science; Member Western Society of Engineers, Society for Promotion of Engineering Education, American Federation of Arts, Etc.

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IN MEMORY OF MY WIFE

MARY CARTTER STEELE RICKER

1852–1875–1910

PREFACE

FOURTEEN years' experience with the State examinations for license to practise architecture in Illinois has clearly shown that applicants are usually able to determine the stresses in the members of roof trusses, but are frequently unable to dimension them accordingly or to connect them properly at the apexes of the trusses. The necessary formulas and data are scattered in numerous reference works, and their selection and exact application require more time than is allowed to the busy architect or engineer.

Therefore the purpose of the present work is to supply the data, methods, formulas, and tables required in the design of a roof, arranged in the simplest manner and so as to require the least time and labor in their application.

Many years spent in the instruction of students have shown the author that the study of a series of selected and fully solved examples is more useful in the training of the student than the mere memorizing of many rules and much good advice without frequent applications to practical cases.

Twenty-five examples of roofs are treated in successive chapters, each chapter being confined to a single stage in the process, such as determining stresses, dimensioning members and detailing connections, estimating weights of trusses, etc.

Only the methods of graphostatics required for roof trusses are first given, followed by a description of the parts of wooden and steel roofs, succeeded by the data and formulas required for computing loads on roofs. New formulas are given for the approximate weights of roof trusses, based on the careful design and calculation of numerous examples. It may surprise some readers to learn that steel trusses may be safely designed considerably lighter than those chiefly constructed of wood. An original formula is given for weight of snow, applicable to the United States, excepting the Pacific States. A simple formula for wind pressure must suffice until the actual conditions have been determined by extended experiments and research.

Calculations are made and stresses obtained and tabulated for the selected types of trusses, assuming the maximum snow and wind loads to occur together, as generally assumed in practice. But the author believes that this condition scarcely ever occurs, and he has assumed in Chapter XI that either snow or wind with the permanent loading produces the maximum stress in the members of the truss.

About 100 typical roof trusses have been selected from American, English, French, and German technical works as likely to occur, and their truss and stress diagrams are given in Chapter V.

It is very convenient and often necessary to check the results obtained by the graphical method, and therefore the method of moments is applied to one truss.

In making working or shop drawings, the centre lengths of members must be accurately computed to the nearest 1/32 inch: the methods of calculation employed are illustrated by applications to triangular and cylindrical roofs, and finally to a hemispherical ring dome.

Wind pressure on roof and walls necessarily affects the stability of the walls supporting the roof, though this is often neglected. But several examples are treated, including an open train shed and an enclosed steel-frame building.

The author believes that the subject of the strength of materials may be placed in a far simpler form, more easily applicable to practical cases, by changing loads and safe coefficients of resistance from pounds to tons, and lengths from inches to feet. Also, by inserting in the formulas the safe stresses and safe limited deflections, these formulas are put into a form requiring only very simple computations. Some novel formulas are also provided, especially for members under compound stresses. Others are also given for designing the connections at apexes.

In order to save time and labor for the busy man, a series of tables have been devised, numerical and graphical, by which the most economical safe dimensions of nearly all members and parts of trusses may be determined by simple inspection without calculations.

These are followed by chapters on dimensioning members and detailing connections, with applications to several examples selected from Chapter IV.

After the erection of a truss, the application of its loads slightly changes its form by shortening members in compression and extending those under tension. The method of computing these changes and those caused by changes in temperature is illustrated, thus affording a means of slightly changing the original lengths of members in order that the truss may take the desired form, when finally erected and loaded.

It is often necessary to compute the weight of a roof truss, then to compare this with its assumed weight, in order to correct the stresses, dimensions of members, and their weights, so as to obtain the actual and final weight of the truss. This method of correction is fully illustrated.

Finally, all these preceding methods of treating a truss problem are successively applied to a selected truss in order to show clearly the system of treating a roof recommended for use by practical men.

No part of my "Graphic Statics and Trussed Roofs" (published 1885) has been used in this volume.

My wife aided in making the computations required for the tables in Chapter X.

A large part of the labor required in making the illustrations has been performed by Miss Ethel Ricker, '04, Mr. R. Arnold, '11, and Mr. W. C. Voss, '12.

N. Clifford Ricker.

Urbana, Illinois,
February 2, 1912.

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DESIGN AND CONSTRUCTION OF ROOFS

CHAPTER I

ELEMENTS OF GRAPHOSTATICS

1. Definitions.—The science of statics treats of forces in equilibrium, neither neutralized nor destroyed. It may be studied analytically as in Mechanics, where formulas are deduced for different cases. Or it may be treated graphically, geometrical diagrams being employed for obtaining results similar to those secured by formulas. The analytical method is preferable in treating a general class of problems; the graphical is best for solving a particular problem, especially when unusual conditions occur.

Graphostatics comprises these graphical methods for solving the problems of statics. First developed into a science and published by Culmann in 1866, then introduced in the United States in 1875, it has since been simplified and developed by numerous authors, until it has become indispensable to the architect and engineer. Simple diagrams replace complex calculations by algebraic formulas with the advantages of rapidity, accuracy, easy checking of results, and detection of errors.

2. Representation of a Force.—A force may act at any point in its straight line of action. Therefore a force may be graphically represented by a straight line, in its magnitude, location, and direction.

The unit of magnitude of force may be any unit of weight, usually the pound or the ton of 2,000 pounds; the number of weight units in the force being represented by an equal number of linear units on a straight line, at any convenient scale. In Fig. 1, four tons are represented at a scale of four tons to one inch.

The location of a force is a straight line coincident with the force and drawn through the point at which the force is applied.

The direction of a force may be in either direction along its line of action, being indicated by an arrow-head on line of action.

Forces hereafter considered are always assumed to act in a common plane, represented by the plane of the paper in drawings.

3. Resultant of Forces.—The resultant of any number of forces is that single force that can fully replace all the given separate forces. The antiresultant of those forces is that single force which is in equilibrium with all the separate forces, so that no movement occurs.

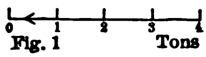


FIG. 1.—A Force.

It therefore follows, that the resultant and the antiresultant of any number of forces must have equal magnitudes, a common line of action, and opposed directions.

4. Couple.—A couple consists of two forces with equal magnitudes, opposed directions, and parallel lines of action. Hence a couple cannot have a single resultant or antiresultant, since it tends to rotate the plane of the forces, and this rotation can only be prevented by another couple with opposed direction and equal moment of rotation.

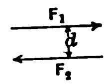


FIG. 2.—A Couple.

5. Equilibrium of Forces.—When any number of forces are in equilibrium, whether acting at a common point or merely in a common plane, the following conditions must exist:

1. The position of the point or plane remains unchanged.
2. The given forces have neither resultant nor antiresultant.
3. The force polygon composed of the representatives of these forces must close.
4. Their equilibrium polygon must also close, if properly drawn with its successive angles lying on the lines of action of the forces taken in the same order, unless their resultant proves to be a couple, which rarely occurs.

6. Composition of Forces.—Several given forces may be combined to obtain their resultant for replacing them, or their antiresultant to equilibrate them. The graphical methods employed vary according to the number and direction of the forces alsc according to whether they are concurrent or not, but these are all primarily based on the principle of the triangle of forces.

That if three forces act at a common point and are in equilibrium, their representatives must always form a triangle, around which the forces have the same continuous direction.

7. By Force Triangle.—Let the two given forces F_1 and F_2 act at the point A in Fig. 3. F_1 is represented by $B\,A$ and F_2 by $C\,A$ in magnitude, location, and direction. Draw $B\,D$ parallel and equal to $A\,C$; join $D\,A$, which represents the resultant R of the two forces F_1 and F_2, because in the force triangle $D\,B\,A$ the forces

F_1 and F_2, have the same direction around it; their resultant connects their beginning and ending, but must have an opposed direction to replace F_1 and F_2 at A. Hence the principle of the force triangle.

But if the direction of their resultant be reversed, it becomes their antiresultant, has their direction around the force triangle, is in equilibrium with them and prevents any displacement of the point A.

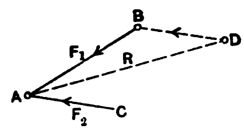


FIG. 3.—Two Forces.

8. By Force Polygon.—Let the four given forces F_1, F_2, F_3, and F_4 act at the point A in Fig. 4. They are here represented by the lines $B\,A$, $C\,A$, $D\,A$, and $E\,A$. Draw $F\,B$ parallel and equal to $A\,C$ and join $F\,A$, thus obtaining R_1, the resultant of F_1 and F_2; combine in like manner R_1 and F_3, obtaining R_2; lastly, R_2 and F_4, producing R_3, the entire resultant R of all the forces.

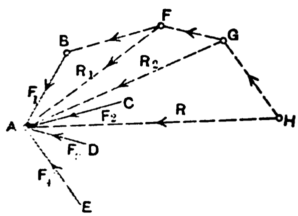


FIG. 4.—Concurrent Forces.

If its direction be reversed to act from A to B, it becomes their antiresultant and is in equilibrium with the given forces at A. This method evidently consists in successive applications of the force triangle.

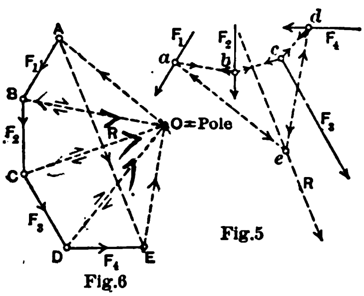
9. By Force and Equilibrium Polygons.—If the lines of action of the given forces neither intersect at a common point nor are parallel, the method of the equilibrium polygon is most convenient for combining the given forces into a resultant.

Let the given forces F_1, F_2, F_3, and F_4 act at different points in a common plane as in Fig. 5, being neither concurrent nor parallel.

The method of the force triangle might be applied to forces F_1 and F_2 by prolonging their lines of action to intersect, obtaining their resultant at this point, then combining their partial resultant R_1 with F_3, proceeding in this manner until the final resultant R_3 is found, fully determined in magnitude, location, and direction. Yet some intersections may fall outside the limits of the paper, so that it is better and equally accurate to use the method of the equilibrium polygon.

In Fig. 6 represent F_1 by $A\,B$, F_2 by $B\,C$, F_3 by $C\,D$, and F_4 by $D\,E$, laid off with the same continuous direction from A to E. Join $A\,E$, which represents the resultant R of the forces in magnitude and

direction; the resultant R in Fig. 5 must also be parallel to $A\,E$ in Fig. 6, but its location is not determined.



FIGS. 5, 6.—Non-concurrent Forces.

Select any point or pole O in Fig. 6 and join it with each angle of the force polygon by the rays $O\,A$, $O\,B$, $O\,C$, $O\,D$, and $O\,E$. In Fig. 5, commencing at any point a on the line of action of F_1, draw $a\,e$ parallel to $O\,A$; $a\,b$ parallel to $O\,B$ and intersecting the line of action of F_2 at b; similarly $b\,c$ parallel to $O\,C$, $c\,d$ parallel to $O\,D$, and $d\,e$ parallel to $O\,E$.

10. Demonstration.—At a act the force F_1, the force $B\,O$ from a toward b, and the force $O\,A$ from e toward a. These three forces must be in equilibrium at a, since their representatives in Fig. 6 form the force triangle $A\,B\,O$ with a continuous direction around it. For the same reason, the forces F_2, $C\,O$, and $O\,B$ are in equilibrium at b; F_3, $D\,O$, and $O\,C$ at c; F_4, $E\,O$, and $O\,D$ at d.

But the forces acting along the side $a\,b$ must be equal, each being represented by $B\,O$ in Fig. 6, and the directions at the ends of $a\,b$ are opposed; hence the side $a\,b$ is in equilibrium; likewise, for similar reasons, the sides $b\,c$ and $c\,d$ are each in equilibrium. The four forces F_1, F_2, F_3, and F_4 have been equilibrated, and there remain for further consideration only the forces $O\,A$ acting along $e\,a$, and $E\,O$ acting along $e\,d$.

Yet these last forces are also in equilibrium with the resultant R at their intersection e, since with R their representatives in Fig. 6 form the force triangle $O\,A\,E$ and have a continuous direction around it.

Therefore through e, the intersection of the first and last sides of the equilibrium polygon $a\,b\,c\,d\,e$, draw the line of action of the resultant R parallel to $A\,E$ in Fig. 6, which is its required location. The polygon $a\,b\,c\,d\,e$ is termed the equilibrium polygon, because each of its sides is in equilibrium, whatever pole O be selected.

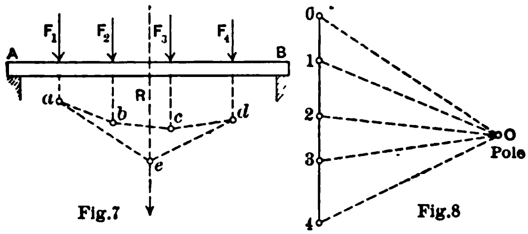
This affords an easy means of checking the accuracy of the work by selecting a different pole O' and repeating the process, obtaining another point e', which must fall on the line of action previously found for R.

This method of the equilibrium polygon should be fully mastered,

since it is a powerful method with wide application in practice.

The given forces are often actually parallel, particularly in the case of horizontal beams, girders, and roof trusses.

11. Application to Parallel Forces.—Let the forces F_1 to F_4 act vertically on a horizontal beam $A\,B$ as in Fig. 7. From left to right, they are successively laid off downward on the vertical line 1 to 5 of the force polygon in Fig. 8, which here becomes a vertical



FIGS. 7, 8.—Vertical Forces.

straight line. With any pole O, draw the rays $O\,0$, $O\,1$, $O\,2$, $O\,3$, and $O\,4$. Beginning at any point a on F_1, draw the equilibrium polygon $a\,b\,c\,d\,e$ as before. The resultant R of the given forces is represented in magnitude and direction by 0 4 in the force polygon of Fig. 8, and it must act vertically through the point e, the intersection of the first and last sides of the equilibrium polygon.

12. Resolution of Forces.—This is the opposite of the composition of forces, meaning that a single force is to be replaced by two components of unknown magnitudes, but whose locations and directions are fixed. Resolution of a force into three or more components is generally indeterminate.

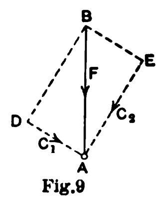


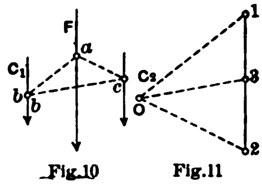
FIG. 9.—Resolution of a Force.

13. Components Oblique to the Given Force.—Let F in Fig. 9 be resolved into the components C_1 and C_2 with prescribed lines of action $A\,D$ and $A\,E$, intersecting the force at A. Complete the force triangle $A\,D\,B$ by drawing $D\,B$ parallel to $A\,E$. Or the entire parallelogram $A\,D\,B\,E$ may be drawn. Then C_1 is represented in magnitude by $D\,A$ and C_2 by $B\,D$ or $E\,A$.

14. Components Parallel to the Force and to Each Other.—Two cases.

The force F lies between the required components, as in Fig. 10. Draw the force polygon in Fig. 11 by making 1 2 equal to F. With

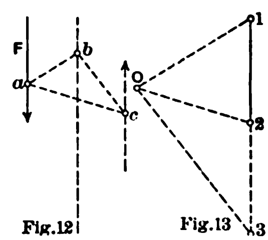
any pole O, draw rays O 1 and O 2; commencing at any point a on F in Fig. 10, draw $a\,b$ parallel to O 1 and cutting C_1 at b; draw $a\,c$ parallel to O 2; join $b\,c$ and parallel to it draw the dividing line O 3, which divides F into the components $C_1 = 1\ 3$ and $C_2 = 3\ 2$.



FIGS. 10, 11.—Force between Components.

The force F may not lie between the required components, as in Fig. 12. This case is solved in the same manner, but the direction of the component C_2 most distant from F will be opposed to that of F; the nearest component C_1 will have the same direction as F, but its magnitude will be the sum of F and C_2.

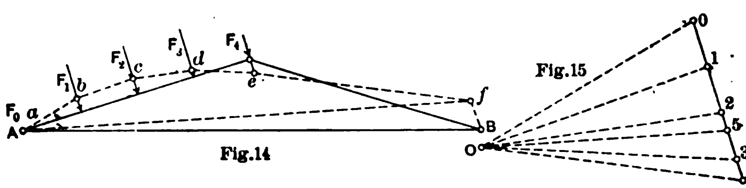
15. Reactions at Ends of a Beam or Roof Truss.—The loads supported by a horizontal beam or roof truss are equilibrated by opposed reactions at its end supports, the sum of these reactions exactly equalling the antiresultant of all loads supported, including the weight of beam or truss. This principle is applicable to any form of roof truss forming a connected structure. These end actions R_1 and R_2 must be found before commencing to draw the stress diagrams for a roof truss. There are two general cases.



FIGS. 12, 13.—Force outside Components.

16. Ends Both Fixed.—Both ends of the truss are fastened to the walls or supports. Since the expansion of a steel truss caused by heat is relatively small, trusses having spans of 100 ft. or less are usually fixed to the walls.

Let Fig. 14 represent the outline of such a truss, supporting the



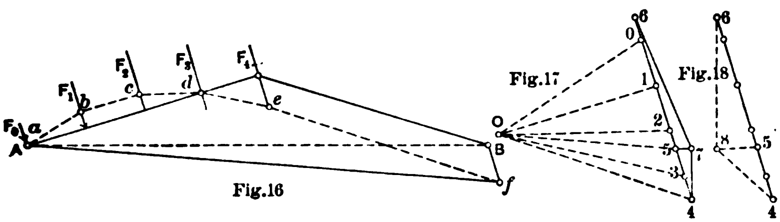
FIGS. 14, 15.—Reaction at Ends of Beam.

inclined parallel (wind) loads F_1, F_2, F_3, and F_4. Laying off these loads from left to right successively on the parallel line O 4 in Fig. 15, with pole O draw the rays O 0 to O 4. Then the polygon $a\,b\,c\,d\,e\,f$

in Fig. 14, intersecting at f the line Bf, parallel to $O\,4$ in Fig. 15. Join Af by closing line Af, and parallel to Af in Fig. 15 draw dividing line $O\,5$, which divides the antiresultant of the forces into $5\,0$ = resultant R_1 at A, and $4\,5$ = resultant R_2 at B, which are the required end reactions for the given truss and loads. The application of this method to vertical loads or a horizontal beam is precisely similar.

For steel trusses of more than 100 ft. span, a slip plate or expansion rolls are usually placed under one end of the truss to prevent the walls from being forced out of a vertical position.

17. Expansion Rolls at Windward.—First assume that the rolls are placed at the left end A. The force and equilibrium polygons are drawn as before in Figs. 16 and 17, obtaining the closing line Af and the dividing line $O\,5$. Extend the load line upward by the half load acting directly at A. For sake of clearness, the points 6, 5, and 4 are transferred to the parallel load line in Fig. 18. If the rolling friction of the rolls be neglected, being very small in proportion to the reaction at A, only a vertical reaction could be transmitted from the wall through the rolls to the truss at A. Hence



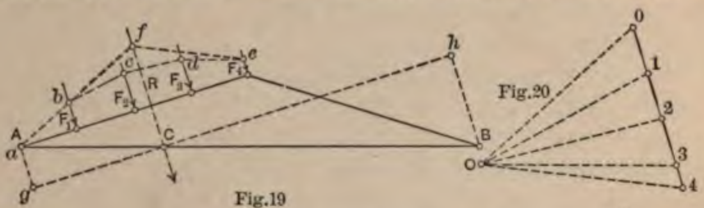
FIGS. 16–18 —Rolls at Windward or Leeward.

draw the vertical $6\,8$ to intersect a horizontal through the dividing point 5; join $8\,4$. Then at A, $R_1 = 8\,6$, and at B, $R_2 = 4\,8$. In case of a slip plate, the friction resistance to sliding can be computed and laid off as a horizontal, the actual reaction R_1 then being the hypothenuse of a triangle composed of this horizontal and the vertical reaction previously obtained.

18. Expansion Rolls at Leeward.—Next assume the rolls to be placed under the right end B. In Fig. 17, draw the vertical $4\,7$ through 4 to intersect a horizontal through the dividing point 5; join $7\,6$. Then at A, $R_1 = 7\,6$, and at B, $R_2 = 4\,7$.

Neither rolls nor slip plates are required for large trusses with wooden tie-beams and chiefly constructed of wood.

19. Another Method.—These end reactions may also be found by a different graphical method, to serve as a useful check on that just explained. Resuming the roof truss and loads of Fig. 14 as represented in Fig. 19, its force polygon in Fig. 20 and equilibrium polygon in Fig. 19 are drawn as before, excepting that the first and



FIGS. 19, 20.—Another Method.

last sides of the equilibrium polygon are extended to intersect at F, through which the resultant R of the loads is then drawn to cut the horizontal $A\,B$ at C. Draw any straight line through C, $a\,g$, and $h\,B$ parallel to resultant R. Then by similar triangles, $A\,g\,C$, $B\,h\,C$:

$$R\frac{C\,B}{A\,B} = \text{reaction } R_1 \text{ at } A \quad . \quad . \quad . \quad . \quad (1)$$

$$R\frac{A\,C}{A\,B} = \text{reaction } R_2 \text{ at } B \quad . \quad . \quad . \quad . \quad . \quad (2)$$

20. Moments of Forces.—The pivot of a moment is a fixed point about which the moment tends to rotate the plane of the forces.

The lever arm of a force or moment is the perpendicular drawn from the pivot to the line of action of the force.

The moment of a force numerically equals the product of its magnitude by its lever arm, and it is usually expressed in inch-lbs., inch-tons, foot-lbs., or foot-tons, according to the units of length and weight used.

The direction of rotation of a moment may be positive (watch-wise) or negative (non-watchwise), represented by the signs + and −.

21. Moment of a Couple.—The lever arm of a couple is always the perpendicular distance between the lines of action of its two forces, without regard to the position of the pivot. Hence the moment of a couple has a constant numerical value for all positions of the pivot, equal to the product of one force by lever arm of the couple. For in Fig. 21, the moment of F_1 about the pivot $C = -F_1 \times a\,c$, and the moment

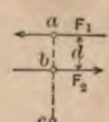
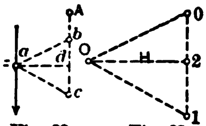


FIG. 21.—Moment of Couple.

of F_2 about $C = + F_2 \times b\,c$. The algebraic sum of these moments $= - F_1 \times a\,b$ (3). The negative sign of the moment indicates that its rotation is negative or non-watchwise. Hence the moment of a couple must always be as previously stated.



FIGS. 22, 23.—Moment of Force.

22. Moment of a Force.—Assume the force F and the pivot A, as in Fig. 22. Draw the force polygon in Fig. 23 as before; then in Fig. 22, draw $a\,b$ parallel to $O\,0$ and $a\,c$ parallel to $O\,1$, $b\,a\,c$ then being the equilibrium polygon. Through pivot A draw $A\,b\,c$ parallel to the force F, and through O and a, $O\,2$ and $a\,d$ perpendicular to the force F. By similar triangles, $b\,a\,c$ and $0\,O\,1$:

$$0\,1 : O\,2 :: b\,c : a\,d.$$

$$\text{Hence } 0\,1 \times a\,d = O\,2 \times b\,c.$$

$$\text{But } H = O\,2 \text{ and } F = 0\,1.$$

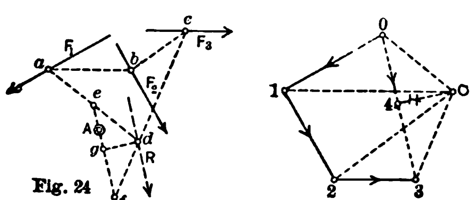
$$\text{Therefore } F \times a\,d = H \times b\,c.$$

But $a\,d$ is the lever arm of F about pivot A. Hence the moment of F about A is equal to $H \times b\,c$. The distance $H = O\,2$ and is termed the "*pole distance.*" The line $b\,c$ is called the "*intercept*" through A.

Hence the moment of a force = pole distance × intercept . (4)

This principle was discovered by Culmann. It has very important and extended applications in practice and therefore should be fully mastered.

23. Moment of Resultant of Several Forces.—The moment of each force about the same pivot might be computed, and the algebraic



FIGS. 24, 25.—Resultant Moment of Forces.

sum of these moments would be the moment of their resultant. This method may be employed as a check on that of Culmann, but the latter will be found more convenient in practice, because it re-

quires little computation, is less liable to error, which is easily detected.

Assume that the forces F_1, F_2, and F_3 are not parallel, as in Fig. 24. Draw their force polygon in Fig. 25, obtaining the resultant $R = O\,3$; draw pole distance H perpendicular to $O\,3$ at 4 In Fig. 24, draw equilibrium polygon $a\,b\,c\,d$; draw through the intersection d of its first and last sides the resultant R parallel to $O\,3$ of Fig. 25. Then draw through the pivot A, $e\,f$ parallel to R. Then the algebraic sum of all the moments of the given forces about A = the *moment of their resultant* R about A =

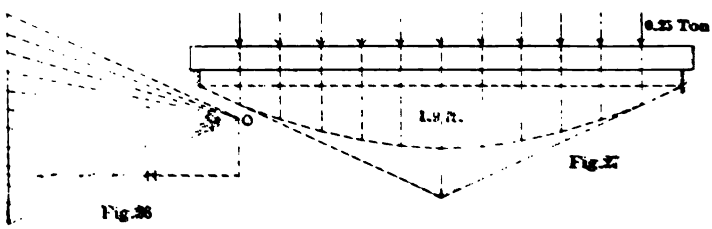
$$\textit{pole distance } H \times \textit{intercept } e\,f. \tag{5}$$

For these forces may be replaced by their resultant R, whose moment about $A = R \times g\,d$. For by similar triangles $e\,d\,f$ and $0\,O\,3$, $R \times g\,d = H \times e\,f$.

Therefore Culmann's principle may be employed for obtaining the resultant moment of any number of forces about a pivot, whether these forces are parallel or not.

24. Bending Moment of Loads on Beam or Girder.—Culmann's principle may be used to determine the transverse bending moment acting at any point in the length of a loaded beam.

Assume a horizontal beam 15 ft. clear length, supporting 11 joists spaced 15 ins. on centres, each transmitting a load of 0.25 ton to the beam. Fig. 27 represents the beam and its loads; Fig. 26 is the force polygon of the loads, amounting to 2.75 tons. Taking



FIGS. 26, 27.—Bending Moment on Beam.

the pole distance H = 12 tons, the equilibrium polygon is drawn in Fig. 27, obtaining a maximum intercept of 1.90 ft. at the middle. Then $M_{max} = H \times$ intercept $= 12.00 \times 1.90 = 22.80$ foot-tons.

The pole distance is measured by the scale of loads and the intercept by that of lengths, employed in the drawings.

The bending moment acting at any other point in the length of

the beam may thus be found by multiplying the intercept at that point by the constant pole distance.

If the pole distance = 1, the bending moment directly equals the intercept. Frequently employed thus by Culmann.

By means of formulas to be given later and the numerical value of M_{max} the required dimensions of the cross-section of the beam may be easily computed for this case, at its middle, ends, and intermediate points in its length.

The end tangents of the equilibrium polygon intersect at f, making an intercept of 3.45 ft., which corresponds to the case in which the same total load is concentrated at the middle of the length of the beam.

Then $M_{max} = 12.00 \times 3.45 = 41.40$ foot-tons.

If the load were uniformly distributed over the length of the beam:

$$M_{max} = \frac{41.40}{2} = 20.70 \text{ foot-tons.}$$

The actual value of M_{max} is about 10 per cent larger than the last, showing that the load should not be assumed to be uniformly distributed in this case, and that the actual value of M_{max} should be found.

25. Reactions by Culmann's Principle.—Resuming the roof truss and loads in Fig. 14, draw its force and equilibrium polygons as in

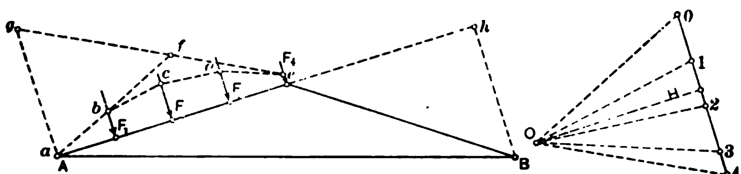


Fig. 28 Fig. 29

FIGS. 28, 29.—Reactions at Ends.

Figs. 28 and 29. Prolong the last side of the equilibrium polygon to intersect at g the parallel to resultant $O\,4$ in Fig. 29 and drawn through A as pivot. In Fig. 29, draw H perpendicular to $O\,4$. Taking A as pivot, the moment of the given forces about $A = H \times ag$.

$$\text{Then } \frac{H \times a\,g}{a\,b} = \text{reaction at } B.$$

$$O\,4 - \text{reaction at } B = \text{reaction at } A \quad . \quad . \quad . \quad (6)$$

This may be used as a check on the method previously given.

CHAPTER II

CONSTRUCTION OF A TRUSSED ROOF

26. Definitions.—The external surface of the roof may be plane, gable, mansard, cylindrical, domical, etc. Its structure may be visible in the interior of the building or may be concealed by a ceiling. One or more stories of rooms are sometimes arranged within the roofs of very large buildings.

All roofs for large structures must be supported by trusses, or by internal partitions, walls, columns, or girders.

A truss is a framework constructed in a vertical plane for supporting the roof and ceiling, sometimes also the floors of lower stories. It must be composed of triangles, since this is the only geometrical form permanently fixed by the lengths of its sides.

The span of a truss is the horizontal distance between its end centres.

The rise of a truss is the vertical distance from its span line to its centre at the upper apex.

A bay of a roof comprises its volume between the vertical middle planes of two adjacent trusses.

A panel of a truss consists of that portion of its elevation between the centre lines of its upper and lower chords, and those of two adjacent vertical or radial web members.

An apex of a truss is the intersection of the centre lines of a chord and of the web members at a common point. The loads supported by a truss are assumed to be concentrated at its apexes, usually only at those of the upper chord, unless a ceiling is attached to the lower chord, or it is loaded by a floor within the roof.

An apex area is that portion of the roof surface, or of a ceiling, supported at a loaded apex of the truss.

A member of a truss connects two adjacent apexes; it may be curved but is usually straight.

The upper chord comprises the entire series of the upper members of the truss, and it is usually a straight, broken, or curved line. When composed of two straight lines, each of these is termed a principal.

The lower chord comprises the lower members of the truss, and it may also be a straight, broken, or curved line. When it is a horizontal timber, this is called a tie-beam.

A web member connects an apex in each of the two chords. The terms radial, vertical, diagonal, etc., are applied to web members.

A strut is a web member only resisting endwise compression.

A tie is a web member only resisting endwise tension.

A strut-tie usually resists compression but sometimes tension.

A tie-strut usually resists tension but sometimes compression.

These alternations of the kind of stress in a member are caused by irregular loading, by heavy snowfall, or wind pressure on one side of roof.

27. Connections at Apexes.—Joints or connections of members in wooden trusses are usually made by properly framing together the wooden members, through which the steel rods usually extend, the joint being often strengthened by bolts, straps, etc.

Pin joints in steel trusses are made on cylindrical pins, inserted in eyes in the ends of the members.

Riveted joints in steel trusses are made by inserting a gusset plate between the two shapes composing each member at the connection, which are then joined to the gusset plate by through rivets. The gusset plate then becomes the common connection of the members.

28. Splices in Members.—A splice in a wooden timber, subject to compression only, should be made as near an apex as possible. A short halved and bolted splice is sufficient and reduces the strength of the timber very little.

A splice in a timber in tension may be made at any convenient place. It is best and simplest to butt together the ends of the timbers, then connecting them by fish plates of steel or hard wood placed on opposite vertical sides of the timbers, strongly bolted through the plates and timbers. A good splice of this kind will safely resist 50 per cent of the safe tensile strength of the uncut timber. Complex forms of splices are not as good and are more costly. Such splices in lower chords are often made by constructing the chord of planks set on edge and strongly spiked and bolted together, their ends being arranged to break joints. Such a chord looks better being of uniform size, but it is very difficult to prevent the planks from sliding on each other. Nor should it be stressed to over 50 per cent of the safe strength of the uncut planks.

A splice in a steel member is usually made at an apex by means of gusset and cover plates, all firmly riveted with through rivets.

29. Elements of a Trussed Roof.—

- Roof enclosure .... { Covering. / Sheathing. / Ceiling.
- Roof framework .... { Rafters. / Purlins.
- Roof truss .... { Upper chord ...... / Web members .... { Struts. / Ties. / Strut-ties. / Tie-struts. / Lower chord .....

DESCRIPTION OF PARTS OF WOODEN TRUSSED ROOF

30. Covering.—This material protects the exterior of the roc from water, snow, and dust, and it is commonly of shingles, c painted tin, or of several layers of felt cemented together and covere

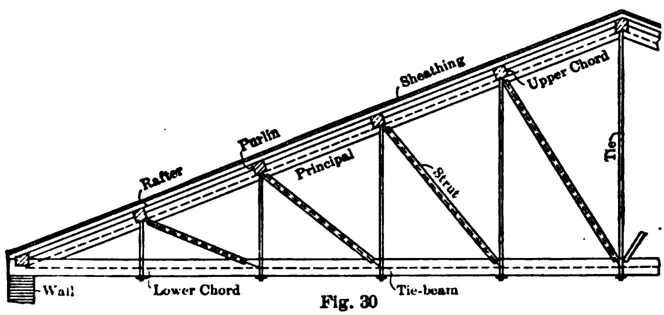


FIG. 30.—A Wooden Truss.

with melted asphalt and gravel. Sheet copper or zinc is sometime employed on costly buildings for greater permanency and to avoi frequent painting. Slates or tiles are often used for durable roofs Duck cloth is also found on the decks of vessels and sometimes o veranda roofs, especially if they are frequently walked upon. I must be kept well painted to prevent rot. Tarred paper or boar is only suitable for temporary sheds, etc.

31. Sheathing.—For shingle roofs, the sheathing is usually rougl pine boards six inches wide, laid with joints open two or three inches Matched and dressed lumber of similar grade is used under all othe coverings, and it should also be covered with ruberoid, heav

the pressure is distributed over the wood by a cast-iron washer of proper area, its face perpendicular to the rod. Skew washers tend to slip and shear off the rod, hence the necessity for a bearing at right angles to the rod.

39. Strut-Ties or Tie-Struts.—The former is under a maximum compression and is subject sometimes to a minimum tension, while the latter is usually under a maximum tension, but is sometimes subject to a minimum compression. The reversed stress may occur under irregular loading or by change of wind from one side of the roof to the other. Therefore the member must be designed to safely resist both its maximum and minimum stresses. It may be a wooden timber connected with the chords at each end by side straps and bolts, by clips and bolts, etc., strong enough to resist the tensile stress. Or it is sometimes composed of two struts set beside each other to resist compression, between them being placed a rod for resisting tension; or two rods may be used beside a single strut.

Reversal of the kind of stress in a chord or web member rarely occurs in a truss of ordinary form, but it must be considered in trusses for high roofs, scissors trusses, and especially in large cylindrical roofs and domes.

40. Diagonals.—Diagonal rods connect alternate apexes of two adjacent trusses in order to strengthen the roof against diagonal winds and accidental loads. Suspension rods for supporting purlins are not used except with steel purlins.

41. Parts of a Steel-Trussed Roof.—The steel roof truss is now more common than those of wood, for reasons of economy, permanence, and appearance. The rafters and purlins are also of steel in important permanent or firepoof structures.

In Europe, and sometimes in the United States also, the members of roof trusses are frequently protected by hollow tiles, or they are now frequently made of reinforced concrete, which will probably soon be common here in fireproof buildings.

42. Provision for Expansion of Truss.—On account of changes in the lengths of steel members caused by variations in temperature, steel trusses of more than 100 ft. span should have rolls or a slip plate under one end, the other being fixed to the wall. Or if both ends are fixed to heavy walls, or are set on joint pins at the floor level, the truss must also be pivoted at the upper apex and arranged to rise and fall there as the temperature varies. In the latter case

nailed, thus being parallel to the upper chord and not horizontal as if rafters were used. This method is probably more economical than rafters cut to the curve, but careful construction is necessary for a tight roof. Yet it has a better appearance internally.

35. Upper Chord.—A straight principal is usually of uniform cross-section for its entire length for sake of appearance. It is best spliced near an apex, if necessary. It may be of either square or rectangular section, but is best if its depth be greatest. When timber is sawn to order and the principal is a single timber, its cross-section might be reduced toward the upper end with economy, since the stresses in it are there much less.

A curved upper chord is usually built of several thicknesses of plank bent in a vertical plane over each other on a proper form, then being strongly spiked and bolted together to prevent slipping. Joints of the planks are lapped. Such a chord is constructed on the floor and then raised to its place in the roof. But the planks slip on each other under stress, the curvature becomes flattened, and the actual safe resistance of such built curved beams cannot be computed with much accuracy or certainty.

A much better method is to make this upper chord of straight members connecting the successive apexes. The purlins may then be boxed or blocked out to the required curve, ready to receive the curved sheathing of a cylindrical roof. No uncertainty then occurs.

36. Lower Chord.—If this be a horizontal timber or tie-beam, it is best made of timbers of uniform cross-section, having the same width as the upper chord and at least two inches deeper, to allow for cutting the fibres in various ways. The ends of the timbers are best butted and fish-spliced together. This tie-beam is also frequently made of planks set on edge and spiked and bolted together. If it be curved in a vertical plane, it is constructed of planks likewise, but this is objectionable for the reasons already given.

37. Struts.—Being subject to lengthwise compression only, these are wooden timbers, their ends cut off square and boxed in the inner sides of the chords. To prevent displacement sidewise, their ends are spiked to the chords, or are held in place by stub tenons, wooden or steel dowels.

38. Ties.—These are always in tension and are usually steel rods, often used in pairs to resist large stresses and to avoid the use of large single rods. They extend through the chords, their ends being upset to make them as strong as the body of the rod, and

supported by wooden or steel rafters and purlins, resting on steel trusses.

45 a. Ceiling.—For fireproof structures, the ceiling may be constructed of hollow book tiles as described for sheathing. But it is more frequently made of plastering on expanded metal or wire netting, supported by small steel channels or bars clipped to the chords or the purlins. Such ceilings will probably soon be made of reinforced concrete, but they would be much heavier.

46. Rafters.—May be of wood as already described, being still cheaper, but are better if made of steel shapes. The channel section is most economical for rafters in shapes of equal strength. Inverted T's and L's are still occasionally employed for light roofs, but are much heavier and more costly than channels of equal strength. To fasten wooden sheathing to steel rafters or purlins, it is necessary to fasten by screws or clips a wooden strip flat on the upper edge of the steel shape, to which the sheathing may then be nailed.

47. Purlins.—These are generally made of steel shapes, single or in pairs, I-, channel-, or Z-bars being used.

A single shape may be used, riveted to top of the upper chord at its crossing. Being much stiffer edgewise than sidewise, the purlin is then supported at several equidistant points in the length by suspension rods parallel to the upper chord, connecting the successive purlins, the upper ends of the uppermost rods passing through the web of the ridge purlin. The number and size of these rods will depend on the length, stiffness, and the magnitude of the loads supported by the purlin. This construction is probably most economical in ordinary roofs.

Or two channels may be used for each purlin, connected together by bolts and separators or by lattice bars, their webs being sufficiently separated to produce the required stiffness sidewise. For very long or heavily loaded purlins, trussed shapes may be used. Belly rods in the plane of the roof might also be used. But it is probably more economical to place the trusses nearer each other in large roofs than to truss very long purlins.

48. Upper Chord.—For trusses of moderate spans, the upper chord is generally composed of two Ls, between which are inserted gusset plates of uniform thickness for connecting the members at the joints. Being usually under compression only, these Ls are to be connected at proper intermediate distances by fillers and through rivets, in order to combine them into a single member.

occurs the three-hinged arch or truss, often used for spans of unusual widths.

43. Covering.—For open train and storage sheds, market halls and temporary structures, when the building has no enclosing walls and is not heated in winter, the internal and external temperatures are practically equal. The roof is then often covered with corrugated steel sheets without any wooden sheathing or rafters, these sheets being fastened to steel purlins by clips and overlapping at one end and one side. The purlins should never be set more than six feet on centres. This makes a durable and economical roof, but it is liable to drip inside, whenever any considerable difference occurs between the internal and external temperatures. A safer method is to lay the corrugated sheets on felt over the usual wooden sheathing, or to line the roof beneath with expanded metal or steel laths and plastering. This prevents any drip and makes a warmer building.

But the usual covering of painted tin, sheet copper or zinc, felt and gravel, etc., are more commonly employed for important or permanent buildings.

44. Sheathing.—Usually made of dressed and matched flooring as described for wooden roofs.

Roofs of fireproof buildings are now frequently covered by a slab of reinforced concrete three or four inches thick, clipped to purlins, the rafters being omitted or set as far apart as the purlins. The external surface is then covered with ruberoid or roofing felt, on which wooden strips are nailed to receive wooden sheathing or slates. It makes an economical and durable roof but is unusually heavy.

45. Hollow Tile Roofs.—The best city fireproof structures are also covered with a layer of hollow book tiles set in cement, then levelled externally by plastering with cement mortar. These tiles are twelve inches long and two to four inches thick, and they are supported by inverted steel T's set as rafters and embedded in mortar between the tiles, so as to be protected from fire and produce smooth external and internal surfaces. Wood strips may be nailed on the tiles to receive wood sheathing for a metal roof, or a felt and gravel roof may be cemented directly to the cement plastering, making a heavy but permanent roof.

For large structures with roofs visible in the interior, such as exhibition halls, armories, etc., wooden sheathing is generally

tarred board or felt, or other impervious material, under metal, slates, or tiles, since snow and rain are frequently driven by wind through the crevices of slates or tiles, and this would cause leaks. If the sheathing be visible in the interior of the building, the under side is dressed and the joints are often beaded. For cheap structures dressed flooring is often inverted for sheathing. The sheathing is sometimes laid diagonally to improve its appearance and strengthen the roof by diagonal bracing in its planes. Such sheathing is usually 7/8 in. thick, there generally being no economy in using a greater thickness. But thicker sheathing is sometimes laid directly on the purlins, omitting rafters, or even on the trusses, omitting purlins. This improves the internal appearance, but is more expensive.

In the mill system of construction, planks four inches thick are laid on the purlins or trusses. This is much more expensive than the ordinary construction, but it burns very slowly and is often required for the roofs of factories, etc., within the fire limits of cities.

32. Ceiling.—In churches or public halls, a ceiling of wood or of plastering is often directly fastened to the under side of the rafters, thus leaving the purlins and trusses visible in the interior. Or this ceiling may be placed entirely below the truss and be supported by the lower chord, which is usually concealed. If the truss has a tie-beam, the ceiling joists are usually set at right angles to it, and their ends are either notched in the timber or hung in stirrups from it.

33. Rafters.—These are small timbers, 2, 3, or 4 ins. thick, and from 4 to 12 ins. wide, set from 12 to 24 ins. on centres. They are parallel to the upper chord, and are fastened on the purlins. For cylindrical roofs, the upper edges of the rafters must be cut or furred out to the required curve to receive the sheathing in its cylindrical form. Roof joists are also set like purlins at right angles to the upper chord on which they are fastened. This produces a serious bending stress on the members of the upper chord in addition to their lengthwise compression, which fact must be considered in determining the sections of these members.

34. Purlins.—These timbers are of rectangular section, their larger side being either set vertical or perpendicular to the surface of the roof. They are jointed and supported at the apexes of the upper chord. Intermediate purlins are sometimes set between the apex purlins, especially in cylindrical roofs and also when rafters are omitted. The upper edges of the purlins are set to the proper curve of a cylindrical roof, the sheathing then being bent over them and

When two Ls are not sufficient, two channels are employed, connected by lattice bars on top and bottom flanges. Plates are objectionable in repainting the truss, though a plate is sometimes placed on the top of the chord for trusses of very wide spans, the bottom flanges being latticed. The channels must be separated sufficiently to make the member equally stiff sidewise and edgewise. The connections at the joints are then usually made with joint pins instead of gusset plates and rivets.

For economy in trusses of great spans, the upper chord is sometimes made of four Ls set at its angles and latticed together on all sides. This construction is indeed economical but is dangerous on account of the probability that the stress in the member is not uniformly distributed over the cross-sections of the Ls.

For cylindrical roofs, the upper chord may be composed of two shapes bent hot to the proper curve, which appears best in the interior. But since the safe resistance of such a curved member is much reduced by its curvature, it is structurally preferable and more economical to use straight shapes to connect the adjacent apexes. The purlins can then be easily boxed or blocked out to the required curve to receive the sheathing, as the rafters are usually omitted or wooden rafters may be cut to the curve and fixed on the purlins

49. Lower Chord.—This is likewise made of two Ls for roofs of moderate spans and has riveted connections as already described, but fillers between the Ls are not necessary. For roofs of wide spans, two channels may be latticed together with pin connections. Four Ls latticed together should not be used, for the reasons previously given.

For chords under very great stresses, it may become necessary to use box girders, but these should be avoided if possible, on account of the repainting and the difficulty in making the connections of the members.

50. Struts.—A single L is sometimes used to resist very small stresses, but this is bad, since the L is weakest diagonally and moments result at the ends.

They are ordinarily made of two Ls connected at intermediate points by fillers and rivets. The sidewise stiffness is often increased by bending the shapes slightly outward, using thicker fillers and longer rivets, in order to make the stiffness at least equal sidewise.

For larger stresses, two channels are likewise connected by fillers

and rivets, or by lattice bars in case of pin joints in the truss. For very great stresses, it may become necessary to add plates to the channels.

51. Ties.—For riveted trusses, these are made of two Ls like struts, *but only being in tension*, they do not require intermediate fillers and rivets. For very light stresses single or double flat bars or round rods with flattened ends may be used. Flat eye-bars are generally used for pin-connected trusses, as they may be closely packed on the joint pins. Eye-rods are also used for light stresses. Turnbuckles or sleeve nuts may be used on rods only.

52. Strut-Ties or Tie-Struts.—These are constructed in the same manner as struts, care being taken in designing to make them safely resist either kind of stress that may occur.

53. Joint Pins.—Usually made of cold-rolled steel shafting cut to required lengths, as no turning is then required. Their ends are furnished with cotter pins or Lomas nuts, since the sidewise pressure against the nut is very small, if the connection has been properly arranged. These pins must be carefully designed to safely resist the maximum shearing, bearing, and bending stresses that actually occur at each apex of the truss. But it is more economical and common to use but a few different sizes of pins in a truss, in order to lessen the danger of errors in manufacture and shipment. Cold-rolled shafting is 1/16 inch less in diameter than its nominal size, so that this is to be remembered in designing pin connections.

54. Adjustments for Rods.—Turnbuckles, clevises, or right and left nuts, are used near the middle of a round or square rod, so that its length may be accurately adjusted during the erection of the truss or later. Their forms and dimensions are standard and may be taken from Cambria, Carnegie, etc. Flat eye-bars cannot be so adjusted and must be made of the exact length required.

55. Diagonal Rods.—Steel rods are set below the apex areas and parallel to the roof surface, joining alternate apexes of two adjacent trusses in order to prevent distortion of the roof by wind. They are usually placed in the end and some intermediate bays of the roofs of long buildings, or under great exposures they may be required in each bay.

56. Suspension Rods.—These are placed parallel to the upper chords, divide the span of the purlin into equal spaces, connect adjacent purlins, the uppermost rod passing through the web of the ridge purlin, which is often set vertically.

CHAPTER III

LOADS ON ROOF TRUSSES

57. Kinds of Loads.—These are of four kinds: **permanent, snow, wind,** and **accidental.**

58. Permanent.—This comprises the total weight of **all structural** materials composing that portion of the roof supported by one truss.

59. Covering and Framework.—Weight in pounds per square foot of inclined surface of the roof.

	Lbs.		Lbs.
Wooden shingles	2	Slates, 1/8 inch thick	5
Painted tin	2	Slates, 3/16 inch thick	7
Painted sheet steel	3	Slates, 1/4 inch thick	10
Corrugated sheet steel	4	Tiles, flat	15
Copper or zinc sheets	2	Tiles, Spanish	9
Lead sheets	6	Tiles, Ludovici	8
Ruberoid	2	Tiles, Celadon	8
Felt and asphalt	3	Tiles set in mortar, add	10
Felt, asphalt, and gravel	6	Glass in skylights	5

60. Sheathing.—

	Lbs.		Lbs.
Wood, soft, per inch thick	3	Tile arches, 6-inch	25
Wood, hard, per inch thick	4	Tile arches, 8-inch	30
Tiles, porous, for slating	10	Tile arches, 10-inch	35
Tiles, book	20	Tile arches, 12-inch	40
Reinforced concrete, per inch	12.5	Tile arches, 15-inch	50
Cinder concrete, per inch	10	Mortar, per inch thick	10

61. Rafters.—

	Lbs.		Lbs.
Wood, soft	3	Steel shapes	4
Wood, hard, and L. L. pine	4		

62. Purlins.—

	Lbs.		Lbs.
Wood, supporting rafters	3	Steel, supporting rafters	3
Wood, supporting sheathing	4	Steel, supporting sheathing	4

63. Ceiling.—Per sq. ft. of visible surface.

	Lbs.		Lbs.
Wood ceiling joists	2	Sheet steel on wood sheathing	5
Steel ceiling joists	2	Reinforced concrete, per inch	12.5
Plastering and lathing	10	Tiles, book, plastered	25
Wooden ceiling, per inch thick	3	Tile arches, 6-inch, plastered	32

64. Truss.—The weight of the truss varies with its span and rise, with the distance between centres of adjacent trusses, and further with the intensity of the snow and wind loads, that must be supported by the roof.

The formulas here given result from the careful designing and calculation of the actual weights of a large number of trusses of longleaf pine and steel rods, and of those entirely constructed of steel shapes with riveted connections, for spans increasing from 20 to 200 ft., for rises from 1/10 to 1/4 the span, and for distances between centre planes of trusses increasing from 10 to 30 ft. They may therefore be assumed to closely approximate the actual weights of roof trusses within the given limits, required to safely support the sum of permanent, snow, and wind loads on the roof.

The weight of the truss is given in lbs. per sq. ft. of the area of the horizontal projection of that portion of the roof supported by one truss.

$$w = \frac{\text{span}}{25} + \frac{\text{span}^2}{6200} = \text{weight for wooden trusses.} \quad (7)$$

$$w = \frac{\text{span}}{25} + \frac{\text{span}^2}{12600} = \text{weight for steel trusses.} \quad (8)$$

65. Snow Load.—Also given in lbs. per sq. ft. of the horizontal projection of the roof.

Very little data are available relating to the maximum depth of snow occurring at different localities in the United States. Wherever this depth is known, its weight may be assumed to be 7.8 lbs. per sq. ft. for ordinary solid snow, not saturated with water and frozen hard, per ft. in depth.

Meantime the following empirical formula is proposed for the United States, until the maximum snowfall has been determined at a sufficient number of points for constructing a general formula. The results obtained by this formula would be excessive for the Pacific Coast, where the snowfall is usually light and not cumulative.

Let L° = north latitude of the place.

$$2.5\,(L^\circ - 35^\circ) = \text{maximum weight of snow at the place.} \quad (9)$$

This formula assumes 0 lbs. at Memphis and 35 lbs. at the extreme northern limit of the United States.

66. Wind Load.—The intensity of wind pressure on a plane at right angles to its direction increases with its velocity. The pressure per sq. ft. is also found to slightly diminish as the area of the

plane is increased. This maximum wind pressure may be safely taken as follows:

30 lbs. per sq. ft. for ordinary exposures. . . . . . . (10)

40 lbs. per sq. ft. for medium exposures. . . . . . . (11)

50 lbs. per sq. ft. for maximum exposures. . . . . . . (12)

The first value is generally used for buildings in cities, partially sheltered by other structures.

The second should be used for large isolated buildings.

The third is to be employed for buildings in very exposed locations, on high hills or mountains, on the lake or seashore, etc.

The horizontal pressure of the wind divides into two components, one acting at right angles to the surface of the roof, the other only causing friction on the roof and being usually neglected. Numerous formulas have been proposed for determining the magnitude of the first component, but their results vary greatly, showing that this matter is not properly understood.

Hutton's formula is still chiefly used in the United States and England, but it is complex in form and was based on a series of entirely inadequate experiments by its author, made in the eighteenth century.

Until an accurate formula has been constructed after a long series of careful experiments, the use of the following formulas is recommended in practice.

Let w = wind pressure perpendicular to roof surface in lbs. per sq. ft., i° = inclination in degrees of surface from a horizontal.

$$w = \frac{2}{3} i^\circ \text{ for ordinary exposures.} \qquad (13)$$

$$w = \frac{8}{9} i^\circ \text{ for medium exposures.} \qquad (14)$$

$$w = \frac{10}{9} i^\circ \text{ for maximum exposures.} \qquad (15)$$

The values of w respectively become equal to 30, 40, or 50 lbs. per sq. ft., when i° equals or exceeds 45°.

This normal wind pressure on a roof is sometimes resolved into its vertical and horizontal components, assuming the first to be merely a vertical load on the truss and entirely neglecting the horizontal component. But this procedure is manifestly unsafe.

67. Accidental Loads.—These may or may not be present, and they vary according to the uses of the structure. It is necessary to

consider them in each particular case. The roof may be required to serve as an outlook for a crowd of people, a roof garden or theatre; there may be a line of overhead shafting supported by the trusses and with numerous belts to machines on the floor; a travelling crane and its maximum load may be suspended from the trusses; the top of a derrick in a foundry may be fastened to a truss; a water tank may be placed on the roof; the space within the roof may be utilized for rooms, shops, or storage, etc.

CHAPTER IV

STRESSES BY GRAPHOSTATICS

It first becomes necessary to compute the approximate loads to be supported by the roof truss, then drawing the stress diagrams. A system of simple notation will first be given before applying the method to some selected examples of roof trusses.

68. Notation Employed in Formulas.—

A = inclined apex area in sq. ft., supported at an apex of the upper chord or by an apex purlin.

d = distance in ft. between centre planes of two adjacent trusses.

d' = chord distance in ft. between centre planes of dome trusses or ribs at any apex.

h = rise of truss in ft.

i = angle of inclination in degrees of roof from a horizontal.

l = horizontal panel length in ft.

l' = inclined panel length in ft.

m = number of intermediate spaces between purlins and between apexes.

n = number of spaces between rafters for one bay of roof.

N = number of panels in the truss.

P = permanent apex load in lbs.

S = snow apex load in lbs.

W = wind apex load in lbs.

69. Angle of Inclination of Roof.—

$$\text{Tan } i^\circ = \frac{\text{rise}}{\text{half span}} \text{ for a gable roof of two equal slopes.} \quad (16)$$

$$\text{Tan } i^\circ = \frac{\text{rise}}{\text{span}} \text{ for a shed roof of one slope.} \quad (17)$$

$$\text{Tan } i^\circ = \frac{\text{rise}}{\text{horizontal projection of side}} \text{ for unsymmetrical gable roof.} \quad (18)$$

i° = inclination at apex considered for cylindrical or domical roof.

Bisect each panel of the latter on curve between apexes of upper chord; draw horizontal and vertical through each bisecting point;

these are the horizontal and vertical projections of the curve between end points of any division.

Tan i° at any apex $= \frac{\text{vertical}}{\text{horizontal}}$, corresponding to roof area supported at that apex. This average inclination i° is that of the tangent at the apex and is assumed to be uniform for that apex area. (19)

70. Inclined Panel Length.—

l = horizontal panel length.

$l' = \frac{l}{\cos i^\circ}$ = inclined panel length. . . . . (20)

For cylindrical roofs or domes: compute curved panel length by formula for circumference; compute chord for curved panel length by angle at centre between radii drawn to apexes.

For computing loads on roofs, it suffices to take the lengths of the chords between bisecting points on curve between apexes on truss diagram.

71. Apex Area.—

$A = d \times l$, for roofs with plane surfaces. . . . . . . (21)

$A = d \times$ curved panel length, for roofs with curved surfaces. (22)

But with accuracy sufficient for computing loads in practice.

$A = d \times$ chord between bisecting points on curved chord. (23)

$A = d' \times$ chord between bisecting points on chord or rib of dome. (24)

If absolute accuracy be required for panel areas of domes, the surface areas between meridional planes through centres of trusses or ribs, and horizontal circles drawn through apexes, may be computed by formulas for spherical surfaces; the centres of gravity of these areas may be located and the area be divided accordingly between the apexes adjacent to it; the entire panel area at an apex = sum of parts of adjacent panel areas there supported. But this procedure requires much time and will not usually change the dimensions of truss members obtained by employing the approximate method previously given.

72. Purlin Area.—

$A = d \times l'$, for purlins at apexes of upper chord only. . (25)

$$A = \frac{d \times l'}{m}, \text{ for intermediate purlins, } m \text{ spaces per panel.} \quad (26)$$

73. Ceiling Area.—Computed in the same manner as apex roof areas. The ceiling may be suspended from the apexes of lower chord only, or from intermediate points also. It may further be plane, polygonal, cylindrical, or spherical in form.

74. Calculation of Apex Loads.—

$$P = A(\text{covering} + \text{sheathing} + \text{rafters} + \text{purlins} + \text{truss} \cos i^\circ) = \text{lbs.} \quad (27)$$

$$S = A\ (\text{snow} \cos i^\circ) = \text{lbs.} \quad (28)$$

$$W = A\ (\text{normal wind pressure}) = \text{lbs.} \quad (29)$$

75. Calculation of Total Loads on Half Truss.—For all symmetrical trusses of gable roofs.

$$\text{Permanent load} = P\frac{N-1}{2} = \text{reaction at end of truss in lbs.} \quad (30)$$

$$\text{Snow load} = S\frac{N-1}{2} = \text{reaction at end of truss in lbs.} \quad (31)$$

$$\text{Wind load} = W\frac{N-1}{2} = \text{total wind load on entire truss.} \quad (32)$$

The end reactions are found by the equilibrium polygon.

For irregular or curved trusses, when P, S, and W vary in value, the total loads on the entire truss = sum of loads supported at the apexes.

EXAMPLE 1.—TRUSS WITH VERTICAL RODS

76. Programme.—Type of truss as in Fig. 31: span 100 ft; rise 20 ft.; 10 panels; materials, shortleaf pine and steel vertical rods; covering of painted tin on 7/8-inch sheathing; 2-inch rafters; wooden purlins; trusses, 15 ft. on centres; ceiling plastered and floored, attached to lower chord; location at Chicago, latitude about 42° north; ordinary exposure. Travelling crane and 5 tons.

77. Dimensions.—

$$\text{Tan } i^\circ = \frac{20}{50} = 0.4000 = \tan\ 21.8^\circ = \text{angle of inclination.}$$

$$l = \frac{100}{10} = 10.00' = \text{horizontal panel length.}$$

$$l' = \frac{l}{\cos i^\circ} = 10.77' = \text{inclined panel length.}$$

$$A = d\,l' = 15.00' \times 10.77' = 161.55 \text{ sq. ft.} = \text{apex and purlin area.}$$

78. Apex Loads.—

$$\text{Truss} = \frac{\text{span}}{25} + \frac{\text{span}^2}{6200} = \frac{100}{25} + \frac{100^2}{6200} = 5.61 \text{ lbs. per horizontal sq. ft.}$$

Snow $= 2.5\ (42° - 35°) = 17.5$ lbs. per horizontal sq. ft.

Wind $= \frac{2}{3}\, i° = \frac{2}{3} \times 21.8° = 14.53$ lbs. per inclined sq. ft.

$P = A$ (covering + sheathing + rafters + purlin + truss cos $i°$) $= 161.55\ (2 + 3 + 3 + 3 + 5.61 \cos 21.8°) = 161.55 \times 16.21 =$ 2619 lbs. $= 1.310$ tons.

$S = A$ (snow cos $i°$) $= 161.55 \times 17.5 \cos 21.8° = 2625$ lbs. $=$ 1.313 tons.

$W = A$ (wind pressure) $= 161.55 \times 14.53 = 2347$ lbs. $= 1.174$ tons.

79. Total Loads on Half Truss.—

$$\text{Permanent} = 1.310 \times \frac{N-1}{2} = 1.310 \times 4.5 = 5.90 \text{ tons.}$$

$$\text{Snow} = 1.313 \times \frac{N-1}{2} = 1\ 313 \times 4.5 = 5.91 \text{ tons.}$$

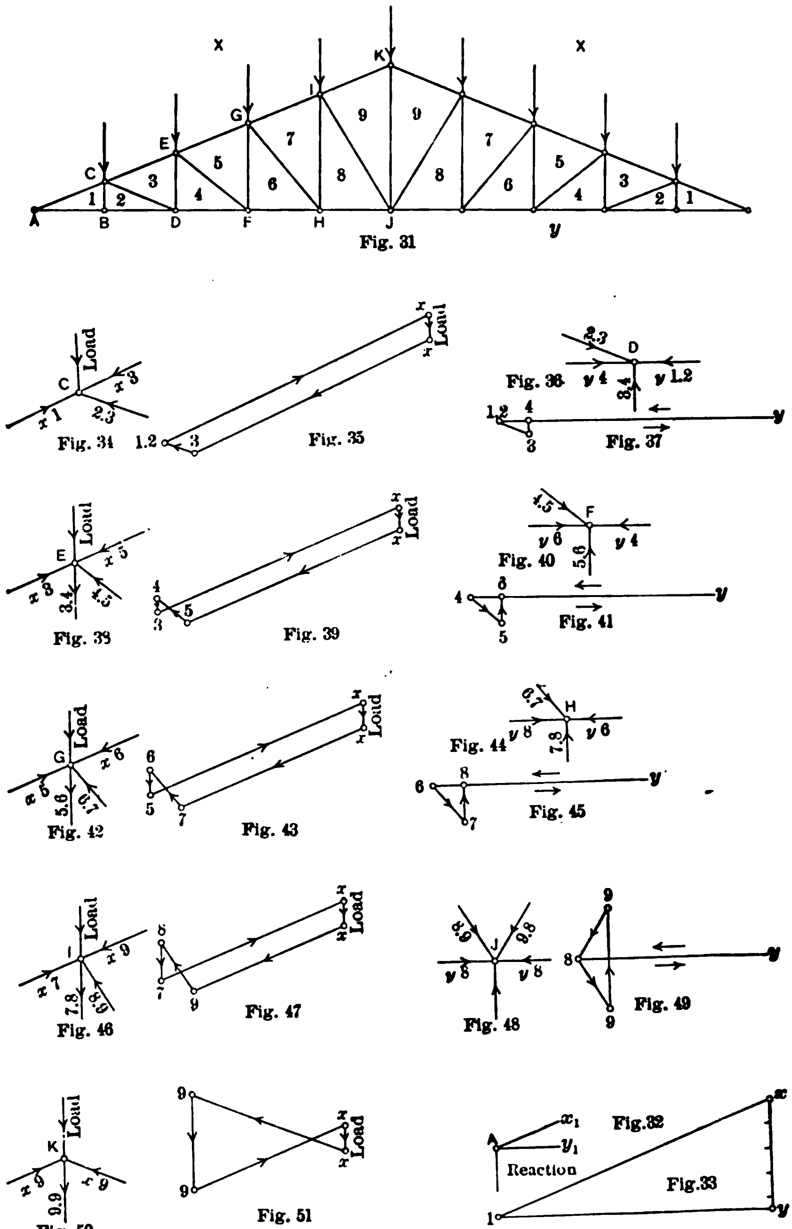
$$\text{Wind} = 1.174 \times \frac{N-1}{2} = 1.174 \times 4.5 = 5.28 \text{ tons on entire truss.}$$

80. Truss and Stress Diagrams.—The truss diagram is drawn at any convenient scale as in Fig. 31, and it is composed of the centre lines of all its members, arranging that these lines may be considerably longer than the corresponding lines in the stress diagrams in order to insure greater accuracy.

81. System of Notation.—The most convenient system of notation is to letter the successive apexes of the truss in capitals from the left end; then to letter the surface above the truss X and the surface below it Y; finally numbering from left to right the successive triangles composing the truss as in Fig. 31. Each member then separates two surfaces in the truss diagram, and its corresponding stress line connects two points lettered like those surfaces. This notation prevents errors in measuring and tabulating the stresses in the members.

82. Stress Diagrams by Culmann's Method.—A separate polygon for permanent stress is then drawn for each apex of the truss.

At the apex A, the upward reaction R must be in equilibrium with the stresses acting in members X 1 and Y 1, Fig. 32. This reaction is one-half the total permanent load on the truss, here being $= 5.90$ tons, as computed. The reaction R is laid off in Fig. 32 from x to y at any convenient scale, and x 1 and y 1 are drawn



Figs. 31-51.—Apex Stress Diagrams.

parallel to X 1 and Y 1 in the truss diagram, Fig. 31. Their magnitudes are to be measured by the scale used for the reaction $R = x\,y$. Since these three stresses are in equilibrium at A, they must have a continuous direction around the stress triangle $x\,y$ 1, and as $x\,y$ acts upward, the stress x 1 must act from x toward 1, and the stress y 1 from 1 toward y, as indicated in Fig. 32.

At apex B, Fig. 31, since no load is at B, no stress occurs in the vertical member 1 2, and the stresses in y 1 and y 2 must then be equal and opposed in direction, as in Fig. 33.

At apex C, Fig. 34, a single load acts downward, which is laid off from x to x in Fig. 35; the magnitude of stress x 1 is known from Fig. 33, and the stresses x 3 and 2 3 are determined by parallels to those members drawn through 1 and the lower x in Fig. 34. The direction of the stresses at C is determined as before.

The stress polygons for the remaining apexes D to K are drawn in the same manner and present no difficulties. They are here illustrated in Figs. 36 to 51. These are permanent stresses only.

83. General Conditions.—The following points should be carefully noted, as they are equally applicable to Cremona's method, soon to be described.

1. Not more than two unknown stresses in members can be determined at any apex, since any method becomes indeterminate for three or more.

2. Comparing the directions of the stresses at the different apexes, these must always be opposed at the ends of any member.

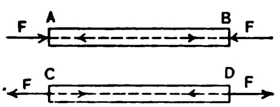


FIG. 52.—Nature of Stresses.

84. Nature of Stress in Member.—3. When the stresses in a member act from its middle toward its ends, the member must be under longitudinal compression, *i.e.*, it is a strut, like the member $A\,B$ in Fig. 52.

4. When the stresses in a member act from its ends toward its middle, it must be under longitudinal tension, *i.e.*, it is a tie, as $C\,D$ in Fig. 52.

Thus the magnitude and nature of the stress acting in any member are both easily determined by the science of Graphostatics.

If the truss diagram be again drawn and the directions of the stresses at the apexes be indicated thereon, the result will be as indicated in Fig. 53. It is evident that compression occurs in the entire upper chord and in the diagonals; tension in the entire lower

chord and the verticals, excepting in 1 2, in which no stress is here found.

85. Stress Diagram by Cremona's Method.—Cremona devised the connected stress diagram, which requires each stress line to be

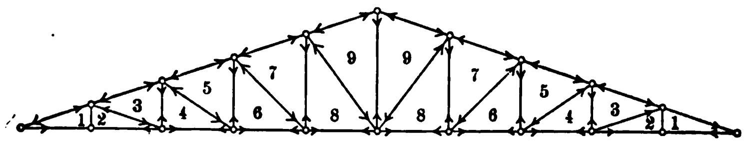


Fig. 53.—Nature of Stresses in Members.

drawn but once, instead of twice, produces a connected diagram, that can be drawn more rapidly and with greater accuracy, because it can be checked in various ways. It is the method now universally used in practice.

The truss diagram and loads of Fig. 21 are repeated in Fig. 54.

Since the permanent and snow stress diagrams must here be symmetrical for both halves of the truss, it is only necessary to draw

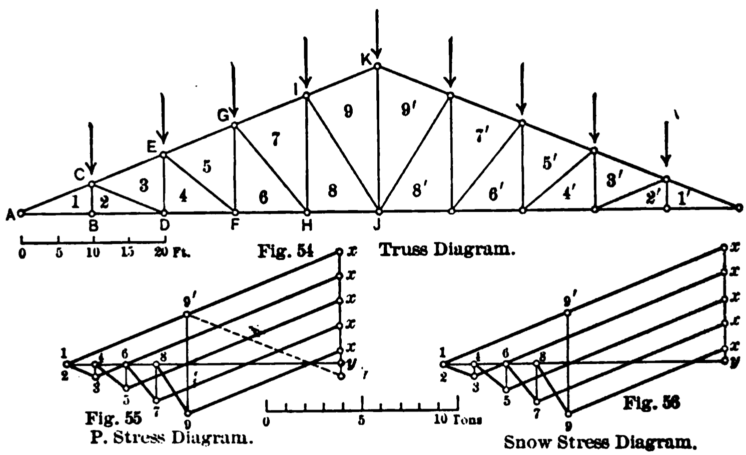


Fig. 54 Truss Diagram.

Fig. 55 P. Stress Diagram.

Fig. 56 Snow Stress Diagram.

Figs. 55, 56.—P. and S. Diagrams.

these stress diagrams for the left half of the truss. Therefore the permanent load for half the truss, or 5.90 tons, is laid off on the vertical $x\,y$ in Fig. 55, which is divided into 4 1/2 equal parts. Commencing at A, the stress lines $x\,1$ and $y\,1$ are drawn parallel to $X\,1$ and $Y\,1$ of Fig. 54, intersecting at 1; the point 2 coincides

with 1, there being no stress in 1 2; through the next x and 2 are drawn x 2 and 2 3, intersecting at 3; through y and 3 are drawn y 4 and 3 4, intersecting at 4, etc. The stress line 9 9′ is here doubled in length because the vertical member 9 9′ in Fig. 54 must support equal stresses transmitted from each half of the truss.

The magnitudes of the stresses may then be measured and their nature determined as by Culmann's method, producing the same results as shown in Fig. 53.

86. Calculation of Snow Stresses.—The snow stress diagram may next be drawn as in Fig. 56, obtaining stresses of identical nature, but of magnitudes differing from those in Fig. 55. Since Figs. 55 and 56 must evidently be similar, the following proportion may be instituted:

$P : S$: : permanent stress : snow stress, or 1.310 : 1.313 : : permanent stress : snow stress in the same member. . . . (33)

Thus the snow stresses may be obtained from the permanent stresses by the use of a good slide rule, or they may be computed by Crelle's or Zimmermann's tables or by logarithms.

87. Wind Reactions and Stresses.—Since the apex wind loads act at right angles to the surface of the roof and on one side only, it is evident that the reactions at the ends of the truss must be unequal, though their sum is equal to the total wind load, and that

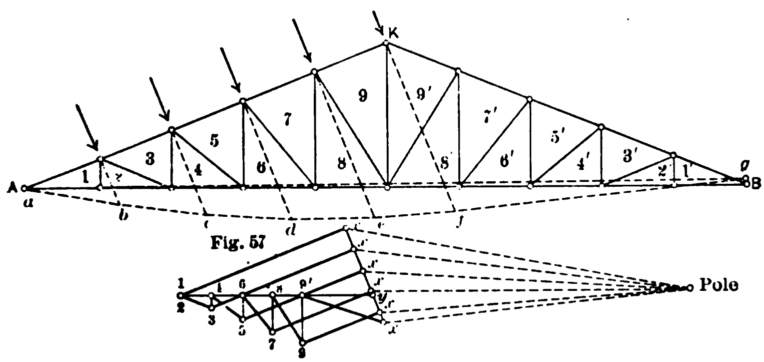


FIG. 58.—Wind Stress Diagram.

this must be divided between the end reactions by the equilibrium polygon, as previously explained in Art. 16, Chapter I.

A half apex load acts at the ridge apex and at the left end apex, but the latter does not affect the stress diagram, unless expansion rolls or a slip plate is used under one end of the truss.

The truss diagram of Fig. 54 is repeated in Fig. 57. The wind loads are laid off in Fig. 58 perpendicular to the principal $A\ K$ of Fig. 57 (Art. 16); by means of the equilibrium polygon is located the dividing point y on load line $x\ x$, the upper reaction $y\ x$ being at A and the lower one $x\ y$ being at B. The stress diagram is then easily drawn; the point 9 here falls on y 1, and x 9 must finally be parallel to the principal on the right side of the truss, as a check on accuracy. Other checks are the following:

88. Checks on Accuracy.—1. For the triangular truss, Fig. 54, the points 2, 3, 5, 7, and 9 lie on a straight line in Figs. 55, 56, and 58.

2. Each alternate vertical cuts the y-line at the same point with an x-line.

89. Ceiling Stress Diagram.—A plastered ceiling is here assumed to be supported by wooden joists, their ends being supported by the wooden tie-beams of the truss. A matched floor is usually laid on the joists to protect the plastering from dust, but it here has no live load upon it.

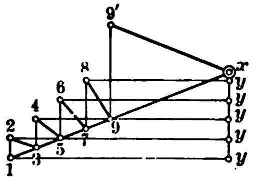


FIG. 59.—Ceiling Stress Diagram.

Then $A \times$ (plastering + joists + floor) $= 150 \times (10 + 3 + 3) = 2{,}400$ lbs. $= 1.2$ tons. Total ceiling load on one-half truss $= 1.20 + 4\ 1/2 = 5.40$ tons.

The stress diagram is then drawn for ceiling of the entire truss as in Fig. 59.

90. Crane Stress Diagram.—Steel I- or channel-beams are sometimes suspended from the lower chords of the trusses at right angles to them, being attached at two apexes near the middle of the truss. A travelling crane then rolls on these beams, on which may be suspended a heavy load at any point of its length. The truss will be most severely stressed when the crane is directly under it, with its maximum load at one end of the crane. This condition is assumed in Fig. 60.

The track beams are suspended at C and G; the crane is 20 ft. long and adds a maximum load of 5 tons, which is assumed to be at C. The reactions at A and B are found by the equilibrium polygon; the larger $= y\ x$ at A and the smaller $= x\ y$ at B. The stress diagram is easily completed as in Fig. 61.

The ceiling and crane diagrams are most conveniently drawn separately, the stresses being entered on the stress sheet for the truss, in order to obtain the total maximum and minimum stresses in the different members.

91. Combined Stress Diagram.—The stress diagrams have been drawn separately, since the method is then more easily understood. But if permanent, snow, and wind loads are to be supported by the roof at the same time, as usually assumed in the United States,

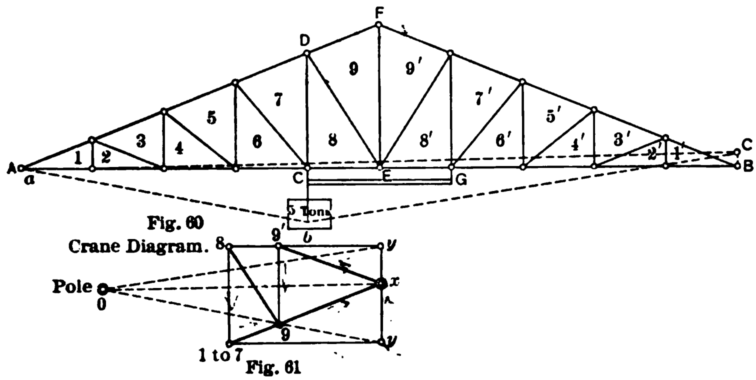


FIG. 61.—Crane Stress Diagram.

producing the maximum stresses in the members, a combined stress diagram may be drawn in most cases to save time. The minimum stresses usually occur under permanent loads alone. They are of the same nature as the maximum stresses, except for high or curved roofs, in which their nature is often reversed.

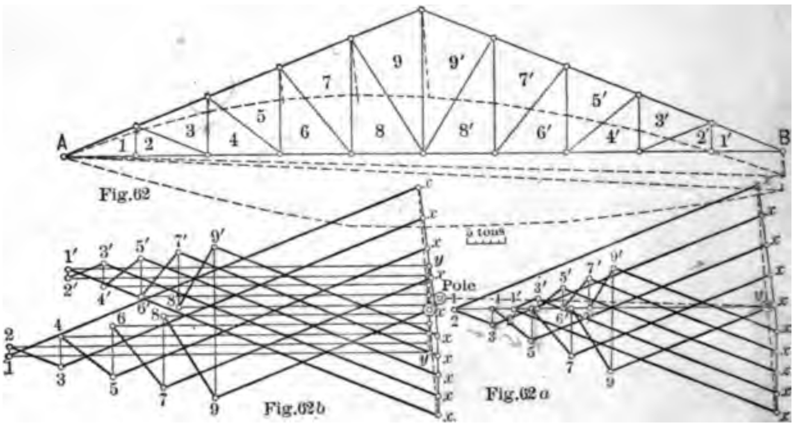
Yet the use of separate diagrams is usually preferable, since the stress sheet always clearly gives the maximum and minimum stresses on each member and indicates a reversal of the nature of the stress.

92. Combined Permanent, Snow, and Wind Stress Diagram.—Resume in Fig. 62 the truss and loads of Figs. 54 and 57. At each apex of the upper chord is laid off the vertical sum of P and S loads; also the W load at right angles to the left principal with a half load at the upper apex; the resultant load at each apex is easily found by the triangle of forces, as indicated.

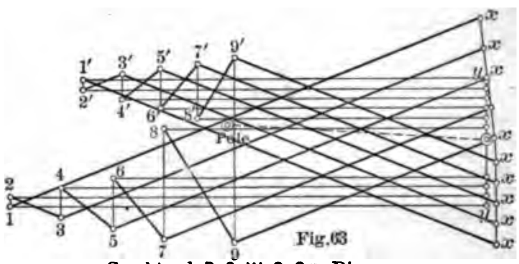
Commencing at A, the successive apex resultants are laid off on the broken load line in Fig. 62 a, whose ends are then joined by a right line, which by the equilibrium polygon is divided at y into the upper reaction at A and the lower reaction at B. The entire stress diagram must be drawn, since the two halves are not similar.

93. Combined Permanent, Snow, Wind, and Ceiling Stress Diagram.—The broken load line in Fig. 62 b is identical with that in Fig. 62 a. After locating the dividing point separating the inclined

end reactions, a vertical line is drawn through this point, on which are laid off the ceiling loads, the middle of the central load being at the dividing point. Then instead of drawing the stress line $y\,1$ through the dividing point as in Fig. 62 a, it is drawn through the lower end of the ceiling load line. The stress line $y\,2$ is drawn



Combined P, S, W, C Diagram Combined P, S, W Diagram



Combined P, S, W, C, Cr. Diagram

FIGS. 62 a, 62 b, 63.—Combined Diagrams.

through the next point on the same line. The completion of the diagram is sufficiently evident.

94. Combined Permanent, Snow, Wind, Ceiling, and Crane Stress Diagram.—This diagram is similar to the last, excepting that the crane load is to be inserted at the proper apex in Fig. 63 in such manner that the two segments of the vertical crane load line may correspond to the end reactions for the ceiling and crane loads. This diagram saves time, but errors are more likely to occur, especially in trusses of complex types.

In any case of combined stress diagrams, it is best to commence at A and draw the diagram for left half of truss, then beginning at B and drawing that for the right half. As a check on accuracy, the middle stress line 9 9′ must be parallel to the corresponding member 9 9′ in Figs. 62 *a*, *b*, 63.

95. Stress Sheet.—After completing the separate stress diagrams, the magnitude of the stress in each member is measured by the scale of the diagram, and the nature of the stress is determined. The stresses are then tabulated on the stress sheet in separate columns for each kind of loading, using the sign $-$ for compression and $+$ for tension. For symmetrical trusses, it is only necessary to include one-half the truss in the stress sheet.

The maximum stress in any member is usually taken by American engineers = the algebraic sum of the $-$ or $+$ stresses acting on it at the same time.

The minimum stress is due to the permanent load alone, excepting on the leeward sides of some types of roofs, where the permanent stresses may be reduced or even reversed by the leeward wind stresses in the member. Both maximum and minimum stresses are easily computed on the stress sheet.

These stress sheets are employed later in dimensioning the members of the same trusses, computing the total weight of the truss, and for correcting stresses in members and their dimensions, so as to accord with any differences between the assumed and actual weights of the truss.

96. Ruled Form for Stress Sheet.—A "cash sales book" bound at the end is very convenient for tabulating the stresses and other data on two adjacent pages; this affords ample space for entering stresses, maximum and minimum stresses, centre lengths and dimensions of members, weights of members and their connections, revised stresses, dimensions, and weights, as will be explained in later chapters.

97. Maximum and Minimum Stresses in Members.—The maximum stress on each member in the following stress sheets is here taken equal to the sum of the stresses due to the P, S, W, ceiling, and crane loads. in accordance with the usual American practice.

But the author regards it as suffiicient to take the larger of the sums of $P + S$ or of $P + W$ stresses as this maximum stress in the member. This maximum is employed in later chapters.

The minimum stress is usually the permanent stress alone.

Note that the maximum stress in X 9 here occurs on the leeward side of the truss. Stresses are always given in tons.

98. Stress Sheet for Example 1.—

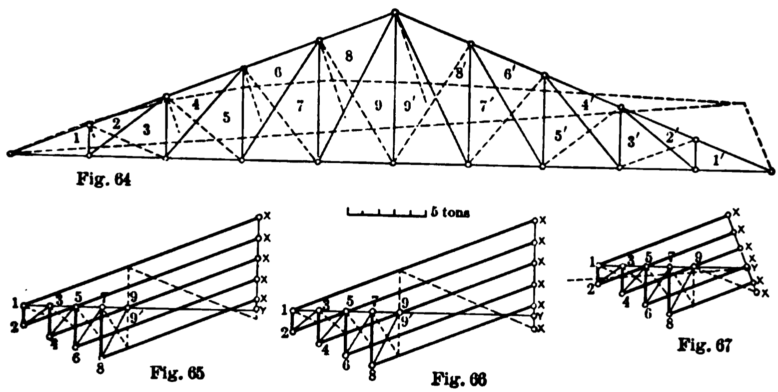
Member.	*P*-stress.	*S*-stress.	*W*-stress.	*Ce*-stress.	*Cr*-stress.	Maximum.	Minimum.
X 2	− 16.6	− 16.6	− 9.0	− 14.7	− 8.0	− 64.9	− 16.6T.
X 3	− 14.3	− 14.3	− 7.8	− 13.1	− 8.0	− 57.5	− 14.3
X 5	− 12.6	− 12.6	− 6.5	− 11.4	− 8.0	− 51.1	− 12.6
X 7	− 10.8	− 10.8	− 5.3	− 9.8	− 8.0	− 44.7	− 10.8
X 9	− 9.0	− 9.0	− 4.1	− 8.1	− 5.4	− 35.6	− 9.0
Y 2	+ 14.9	+ 14.9	+ 9.7	+ 13.7	+ 7.6	+ 60.8	+ 14.9
Y 4	+ 13.3	+ 13.3	+ 8.1	+ 12.2	+ 7.6	+ 54.5	+ 13.3
Y 6	+ 11.7	+ 11.7	+ 6.6	+ 10.6	+ 7.6	+ 48.2	+ 11.7
Y 8	+ 10.0	+ 10.0	+ 5.0	+ 9.1	+ 7.6	+ 41.7	+ 10.0
1 2	+ 0.0	+ 0.0	+ 0.0	+ 0.0	+ 0.0	+ 0.0	+ 0.0
3 4	+ 0.7	+ 0.7	+ 0.6	+ 1.8	+ 0.0	+ 3.8	+ 0.7
5 6	+ 1.3	+ 1.3	+ 1.3	+ 2.4	+ 0.0	+ 6.3	+ 1.3
7 8	+ 2.0	+ 2.0	+ 1.9	+ 3.0	+ 5.0	+ 13.9	+ 2.0
9 9′	+ 5.3	+ 5.3	+ 2.6	+ 6.0	+ 4.0	+ 23.2	+ 5.3
2 3	− 1.7	− 1.7	− 1.7	− 1.6	− 0.0	− 6.7	− 1.7
4 5	− 2.1	− 2.1	− 2.0	− 2.0	− 0.0	− 8.2	− 2.1
6 7	− 2.6	− 2.6	− 2.4	− 2.3	− 0.0	− 9.9	− 2.6
8 9	− 3.1	− 3.1	− 3.0	− 2.9	− 4.9	− 17.0	− 3.1

Example 2.—Truss with Vertical Struts

99. Changes from Example 1.—The truss in Fig. 64 has the same form, dimensions, and loads as that in Figs. 60 and 62, except that its vertical members are to be struts instead of ties. Hence its diagonals are reversed from those of Figs. 60 and 62, *i.e.*, are the other diagonals of the trapezoidal panels of the truss. The truss is further not required to support ceiling and crane loads. This reversal of the diagonals changes the stresses in the verticals from tension to compression and those in the diagonals from compression to tension. This type of truss is then preferable for construction in steel, since the compression members are shorter and their resistances are increased accordingly, making this type more economical for steel. The respective stress diagrams are drawn as before in Figs. 65, 66, and 67, and the stress sheet is then made out in

the same form as before. Compare with Figs. 55, 56, and 57, Art. 85.

100. Comparison of Stress Diagrams for Examples 1 and 2.—Since the dimensions and loads are the same in both trusses, the diagonals alone being changed, the differences between the stress diagrams relate chiefly to the verticals and diagonals, in which the nature and magnitudes of the stresses are changed. In Figs. 65, 66, and 67, the dotted members and stress lines are those of Article 85, the full lines being those of Article 99. For example, in Fig. 65, the trapezoid 1 2 3 4 corresponds to the second trapezoidal panel from left end A in the truss of Fig. 64, with diagonal 2 3 inclined toward



FIGS. 64–67.—Diagonals Reversed.

the ridge of roof, excepting that it is rotated 180°. Hence it becomes evident that one system of diagonals may be changed to the other by merely drawing the other diagonals of the corresponding trapezoids of the stress diagram, without drawing new stress diagrams.

101. Change in Web System and Stress Diagram.—This method of easily changing from one system of diagonals to the other, without the necessity for drawing new stress diagrams, is very important, particularly when a series of radials, verticals, or diagonals is required to be struts or ties only, if possible, especially in trusses with curved chords or for cylindrical roofs. The positions of the diagonals may then be reversed, where necessary.

102. Stress Sheet for Example 2.—Wooden truss.

Member.	*P*-stress.	*S*-stress.	*W*-stress.	Maximum.	Minimum.
X 1	−16.6	−16.6	− 9.0	−42.2	−16.6
X 2	−16.6	−16.6	− 9.4	−42.6	−16.6
X 4	−14.3	−14.3	− 8.2	−37.8	−14.3
X 6	−12.6	−12.6	− 6.9	−32.1	−12.6
X 8	−10.8	−10.8	− 5.7	−27.3	−10.8
Y 1	+14.9	+14.9	+ 9.7	+39.5	+14.9
Y 3	+13.3	+13.3	+ 8.1	+34.7	+13.3
Y 5	+11.7	+11.7	+ 6.5	+29.9	+11.7
Y 7	+10.0	+10.0	+ 4.9	+24.9	+10.0
Y 9	+ 8.4	+ 8.4	+ 3.3	+20.1	+ 8.4
1 2	− 1.3	− 1.3	− 1.2	− 3.8	− 1.3
3 4	− 1.9	− 1.9	− 1.9	− 5.7	− 1.9
5 6	− 2.7	− 2.7	− 2.5	− 7.9	− 2.7
7 8	− 3.3	− 3.3	− 3.2	− 9.8	− 3.3
9 9′	− 0.0	− 0.0	− 0.0	− 0.0	− 0.0
2 3	+ 2.1	+ 2.1	+ 2.0	+ 6.2	+ 2.1
4 5	+ 2.5	+ 2.5	+ 2.5	+ 7.5	+ 2.5
6 7	+ 3.2	+ 3.2	+ 3.0	+ 9.4	+ 3.2
8 9	+ 3.7	+ 3.7	+ 3.6	+11.0	+ 3.7

103. Stress Sheet for Example 3.—Entirely of Steel.

Member.	*P*-stress.	*S*-stress.	*W*-stress.	Maximum.	Minimum.
X 1	−15.8	−16.6	− 9.0	−41.4	−15.8
X 2	−15.8	−16.6	− 9.4	−41.8	−15.8
X 4	−13.6	−14.3	− 8.2	−36.1	−13.6
X 6	−12.0	−12.6	− 6.9	−31.5	−12.0
X 8	−10.3	−10.8	− 5.7	−26.8	−10.3
Y 1	+14.2	+14.9	+ 9.7	+38.8	+14.2
Y 2	+14.2	+14.9	+ 9.7	+38.8	+14.2
Y 3	+12.7	+13.3	+ 8.1	+34.1	+12.7
Y 5	+11.2	+11.7	+ 6.5	+29.4	+11.2
Y 7	+ 9.5	+10.0	+ 4.9	+24.4	+ 9.5
Y 9	+ 8.0	+ 8.4	+ 3.3	+19.7	+ 8.0
1 2	− 1.2	− 1.3	− 1.2	− 3.7	− 1.2
3 4	− 1.8	− 1.9	− 1.9	− 5.6	− 1.8
5 6	− 2.6	− 2.7	− 2.5	− 7.8	− 2.6
7 8	− 3.1	− 3.3	− 3.2	− 9.6	− 3.1
9 9′	+ 0.0	+ 0.0	+ 0.0	+ 0.0	+ 0.0
2 3	+ 2.0	+ 2.1	+ 2.0	+ 6.1	+ 2.0
4 5	+ 2.4	+ 2.5	+ 2.5	+ 7.4	+ 2.4
6 7	+ 3.1	+ 3.2	+ 3.0	+ 9.3	+ 3.1
8 9	+ 3.5	+ 3.7	+ 3.6	+10.8	+ 3.5

It is more convenient to construct the truss in Example 2 entirely of steel shapes instead of wooden timbers and diagonal steel ties. Then $P = 1.249$ and the permanent stresses in the members are reduced in the proportion: 1.310 : 1.249 : : permanent stresses in Example 2 : permanent stresses, if truss is entirely of steel. The stress sheet is then as given for Example 3.

Example 3.—Fink Truss with 8 Panels

104. Description.—The Fink truss in Fig. 68 is frequently employed for steel trusses, because the members in compression are relatively short, all the longer members being in tension. Connections at the joints are easily made by pins or gussets and rivets; the truss appears light and is probably the lightest of all types. But the number of panels in a true Fink truss must be 4, 8, 16, 32, etc.

105. Programme.—Fink type, as in Fig. 68: span 100 ft.; rise 20 ft.; 8 panels; materials, steel truss, shortleaf pine lumber; covering of painted tin on 7/8-inch sheathing; 2-inch rafters; wooden purlins; trusses, 15 ft. on centres; location at Chicago, latitude about 42° north; ordinary exposure; no ceiling.

106. Dimensions.—

$$\text{Tan } i = \frac{20}{50} = 0.4000 = \tan 21.8°;$$

$$l = \frac{100}{8} = 12.50 \text{ ft.}; \; l' = \frac{12.50}{\cos 21.8°} = 13.46 \text{ ft.}$$

$$A = 13.46 \times 15.00 = 201.90 \text{ sq. ft.} = \text{apex and purlin areas.}$$

107. Apex Loads.—

$$\text{Truss} = \frac{100}{25} + \frac{100^2}{12600} = 4.794 \text{ lbs. per horizontal sq. ft.}$$

$$\text{Snow} = 2.5\,(42° - 35°) = 17.50 \text{ lbs. per horizontal sq. ft.}$$

$$\text{Wind} = \frac{2}{3} \times 21.38° = 14.53 \text{ lbs. per inclined sq. ft.}$$

$P = 201.9\,(2 + 3 + 3 + 3 + 4.794 \cos 21.8°) = 3119$ **lbs.** = 1.560 tons.

$S = 201.9\,(17.50 \cos 21.8°) = 3281$ lbs. = 1.641 tons.

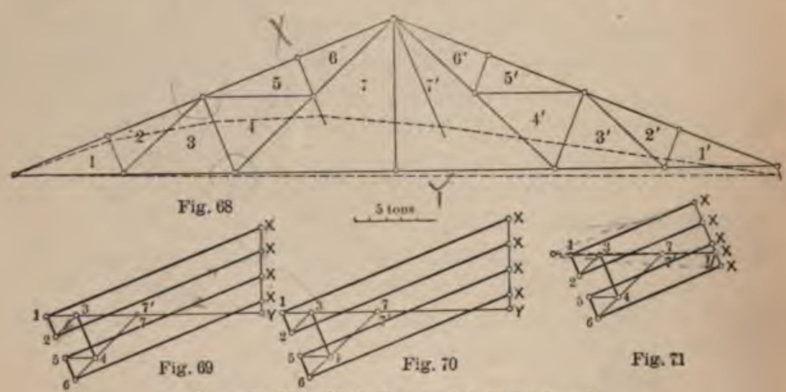
$W = 201.9 \times 14.53 = 2934$ lbs. = 1.467 tons.

108. Total Load on Half Truss.—

Permanent load = 1.560 × 3 1/2 = 5.46 tons.
Snow load = 1.641 × 3 1/2 = 5.74 tons.
Wind load = 1.467 × 3 1/2 = 5.13 tons.

109. Stress Diagrams.—In the truss diagram, Fig. 68, the members 1 2, 3 4, and 5 6 are always to be made perpendicular to the principal. Lower chord is horizontal.

The permanent stress diagram in Fig. 69 presents no difficulty until the member 3 4 is reached. Three unknown stresses act at each apex connected by this member, and the problem at first appears indeterminate. But on inspection of the truss and stress diagrams,



Figs. 68–71.—Fink Truss with Eight Panels.

it is evident that the stresses in the members 2 3 and 3 4 must be equal; also those in 1 2 and 5 6; further that the stress line 3 4 must be drawn parallel to the member 3 4 and also perpendicular to the principal.

If a line 1 6 be traced in Fig. 69 perpendicular to x 1, it is at once evident that the point 5 at the intersection of 1 6 and x 5 is the only point through which 5 4 can be drawn, to equal 2 3 and also be parallel to the member 5 4 in Fig. 68. The points 6, 4, and 7 also lie on a straight line parallel to the members 4 7 and 6 7 in Fig. 68.

The snow stress diagram is entirely similar in form in Fig. 70.

The wind stress diagram in Fig. 71 is drawn in the same manner, after locating the dividing point y on the inclined load line by means of the equilibrium polygon.

The maximum stresses in the members might also be obtained by a combined stress diagram for all the loads on the truss, as shown in Example 1, Fig. 62 a. Or such a diagram might be used for checking the maximum stresses found by computation on the stress sheet.

110. Stress Sheet for Example 3.—

Member.	P-stress.	S-stress.	W-stress.	Maximum.	Minimum.
X 1	−14.7	−15.5	− 8.6	−38.8	−14.7
X 2	−14.1	−14.9	− 8.6	−37.6	−14.1
X 5	−13.5	−14.3	− 8.6	−36.4	−13.5
X 6	−13.0	−13.7	− 8.6	−35.3	−13.0
Y 1	+13.6	+14.4	+ 9.3	+37.3	+13.6
Y 3	+11.7	+12.4	+ 7.2	+31.3	+11.7
Y 7	+ 7.7	+ 8.3	+ 3.3	+19.3	+ 7.7
1 2	− 1.5	− 1.5	− 1.5	− 4.5	− 1.5
3 4	− 2.9	− 3.1	− 3.9	− 9.9	− 2.9
5 6	− 1.5	− 1.5	− 1.5	− 4.5	− 1.5
2 3	+ 2.0	+ 2.1	+ 1.0	+ 5.1	+ 2.0
4 5	+ 2.0	+ 2.1	+ 1.0	+ 5.1	+ 2.0
4 7	+ 4.0	+ 4.1	+ 4.0	+12.1	+ 4.0
6 7	+ 5.9	+ 6.1	+ 6.0	+18.0	+ 5.9
7 7	+ 0.0	+ 0.0	+ 0.0	+ 0.0	+ 0.0

Example 4.—Fink Truss with 16 Panels

This Fink truss of 16 panels would be less economical than one with 8 panels, unless employed for a much larger span than the last, on account of the increased number of connections of the members.

111. Programme.—Fink type of truss, as in Fig. 72: span, 128 ft.; rise, 25.6 ft.; 16 panels; materials, steel; covering of painted tin on 7/8-inch white pine sheathing; rafters of single steel channels; purlins each composed of two channels latticed together; trusses, 16 ft. on centres; location at St. Paul, Minn.; latitude about 45° north; ordinary exposure; rolls under one end of truss.

112. Dimensions.—

$$\text{Tan } i = \frac{25.6}{64} = 0.4000 = \tan 21.8°.$$

$$l = \frac{128}{16} = 8.00 \text{ ft.}; \; l' = \frac{8.00}{\cos 21.8°} = 8.62 \text{ ft.}$$

$$A = 8.62 \times 16.00 = 137.92 \text{ sq. ft.} = \text{apex and purlin areas.}$$

113. Apex Loads.—

$$\text{Truss} = \frac{128}{25} + \frac{128^2}{12600} = 6.43 \text{ lbs. per horizontal sq. ft.}$$

$$\text{Snow} = 2.5\,(45° - 35°) = 25.00 \text{ lbs. per horizontal sq. ft.}$$

$$\text{Wind} = \frac{2}{3} \times 21.8° = 14.53 \text{ lbs. per horizontal sq. ft.}$$

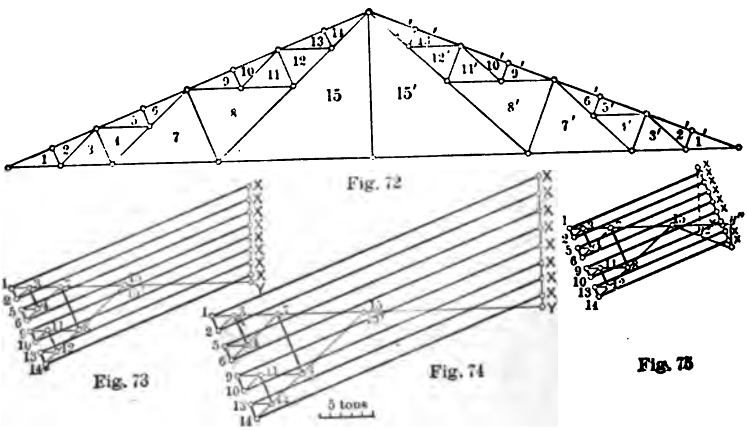
$P = 137.92\ (2 + 3 + 3 + 3 + 6.43 \cos 21.8°) = 2341$ lbs. = 1.171 tons.

$S = 137.92\ (25.00 \cos 21.8°) = 3201$ lbs. = 1.601 tons.
$W = 137.92 \times 14.53 = 2004$ lbs. = 1.002 tons.

114. Total Load on Half Truss.—

Permanent load = 1.171 × 7½ = 8.78 tons.
Snow load = 1.601 × 7½ = 12.01 tons.
Wind load = 1.002 × 7½ = 7.59 tons.

115. Stress Diagrams.—The permanent, snow, and wind stress diagrams are drawn as in Figs. 73, 74, and 75. The line 1 14 is traced perpendicular to x 1 and through 1 in each, and on it lie the stress lines 1 2, 5 6, 9 10, and 13 14. The stress lines 3 4, 11 12, and 7 8 are parallel to 1 14; the lines 6 4 7 and 14 12 8 15 are parallel to the



Figs. 72–75.—Fink Truss with Sixteen Panels.

member 8 15. The completion of the stress diagrams is sufficiently apparent. No stress appears in the vertical 15 15. The stress line 15 x in Fig. 75 must be found parallel to the right-hand principal, as a final check.

116. Changes Due to Expansion Rolls.—In the wind stress diagram, Fig. 75, produce load line upward by half load supported at A; drop vertical through end to cut y-line at y'; erect vertical through lower end of load line to cut y-line at y''. The y-stresses

are to be measured from y', when rolls are at windward end of the truss; from y'', if they are at the leeward end. Hence measure them from y'' to obtain their maximum values. Stresses in other members are not affected by the rollers.

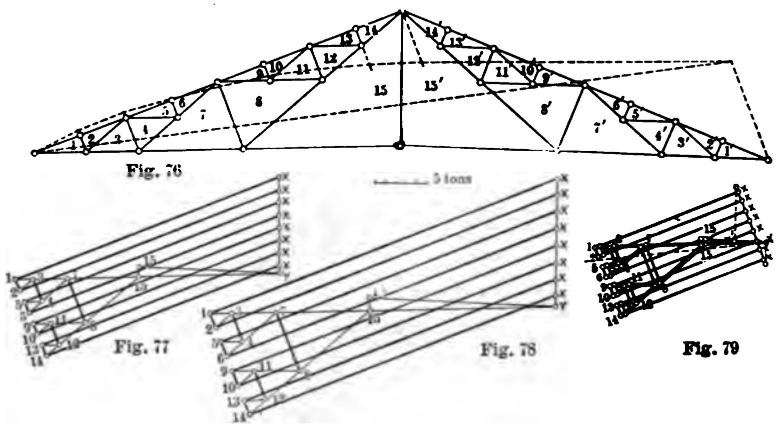
117. Stress Sheet for Example 4.—Rolls at end of truss.

Member.	*P*-stress.	*S*-stress.	*W*-stress.	Maximum.	Minimum.
X 1	−23.9	−32.2	−13.2	−69.3	−23.9 T.
X 2	−23.4	−31.7	−13.2	−68.3	−23.4
X 5	−23.0	−31.1	−13.2	−67.3	−23.0
X 6	−22.7	−30.5	−13.2	−66.4	−22.7
X 9	−22.2	−29.9	−13.2	−65.3	−22.2
X 10	−21.7	−29.3	−13.2	−64.2	−21.7
X 13	−21.3	−28.7	−13.2	−63.2	−21.3
X 14	−20.8	−28.1	−13.2	−62.1	−20.8
Y 1	+22.2	+29.9	+15.0	+67.1	+22.2
Y 3	+20.6	+27.9	+13.7	+62.2	+20.6
Y 7	+17.7	+24.0	+12.0	+53.7	+17.7
Y 15	+11.8	+15.9	+ 5.5	+33.2	+11.8
1 2	− 1.1	− 1.5	− 1.0	− 3.6	− 1.1
3 4	− 2.2	− 2.9	− 2.0	− 7.1	− 2.2
5 6	− 1.1	− 1.5	− 1.0	− 3.6	− 1.1
7 8	− 4.3	− 6.0	− 4.0	−14.3	− 4.3
9 10	− 1.1	− 1.5	− 1.0	− 3.6	− 1.1
11 12	− 2.2	− 3.0	− 2.0	− 7.2	− 2.2
13 14	− 1.1	− 1.5	− 1.0	− 3.6	− 1.1
2 3	+ 1.5	+ 2.0	+ 1.4	+ 4.9	+ 1.5
4 5	+ 1.5	+ 2.0	+ 1.4	+ 4.9	+ 1.5
4 7	+ 3.0	+ 3.9	+ 2.8	+ 9.7	+ 3.0
7 6	+ 5.6	+ 5.9	+ 4.2	+15.7	+ 5.6
8 9	+ 4.5	+ 6.0	+ 4.2	+14.7	+ 4.5
8 11	+ 3.0	+ 3.9	+ 2.8	+ 9.7	+ 3.0
8 15	+ 5.9	+ 8.2	+ 5.5	+19.6	+ 5.9
10 11	+ 1.5	+ 2.0	+ 1.3	+ 4.8	+ 1.5
12 15	+ 8.8	+12.1	+ 8.3	+29.2	+ 8.8
12 13	+ 1.5	+ 2.0	+ 1.4	+ 4.9	+ 1.5
14 15	+10.4	+14.0	+ 9.7	+34.1	+10.4
15 15	+ 0.0	+ 0.0	+ 0.0	+ 0.0	+ 0.0

EXAMPLE 5. FINK TRUSS OF 16 PANELS WITH RAISED LOWER CHORD

Resume the truss and its loads of Example 4, excepting that the lower chord is raised 2.00 ft. at the middle of its length.

118. Stress Diagrams.—The permanent, snow, and wind stress diagrams are drawn as for Example 4, but since the lower chord is



FIGS. 76–79.—Fink Truss with Cambered Chord.

cambered at the middle, a stress will be found in the member 15 15′, as indicated in Figs. 77, 78, and 79.

119. Changes Due to Expansion Rolls.—In the wind stress diagram, Fig. 79, the dividing point on the load line is first obtained by the equilibrium polygon; through this point is drawn a horizontal line, which is intersected at y by a vertical through upper end of load line and at y' by a vertical through its lower end. For rolls at windward end of truss, draw y-lines through y; for rolls at leeward end, draw y-lines through y'. This produces two overlying stress diagrams, as in Fig. 79. The maximum wind stresses in the figure are to be measured and entered on the stress sheet, since the rolls might be at either windward or leeward end of the truss.

120. Effect of Raising Lower Chord.—According to previous investigations by the author, and by comparison of stress sheets 4 and 5, it is at once apparent that the stresses in the upper and lower chords have been materially increased by cambering the lower

chord. The same would be true in a lesser degree, if the rise of the upper chord were at the same time increased, so as to make the depth of the truss at the middle the same in both cases.

Therefore it is more economical to make the lower chord straight and horizontal, there being no gain in cambering it, except by producing a somewhat lighter appearance of the truss.

120a. Stress Sheet for Example 5.—

Member.	*P*-stress.	*S*-stress.	*W*-stress.	Maximum.	Minimum.
X 1	−25.7	−35.1	−14.7	−75.5	−25.7
X 2	−25.3	−34.5	−14.7	−74.5	−25.3
X 5	−24.9	−33.9	−14.7	−73.5	−24.9
X 6	−24.4	−33.3	−14.7	−72.4	−24.4
X 9	−24.0	−32.7	−14.7	−71.4	−24.0
X 10	−23.5	−32.1	−14.7	−70.3	−23.5
X 13	−23.1	−31.5	−14.7	−69.3	−23.1
X 14	−22.7	−30.9	−14.7	−68.3	−22.7
Y 1	+23.9	+32.6	+16.4	+72.9	+23.9
Y 3	+22.3	+30.4	+15.0	+67.7	+22.8
Y 7	+19.0	+25.9	+12.2	+57.1	+19.0
Y 15	+12.7	+17.0	+ 6.2	+35.9	+12.7
1 2	− 1.1	− 1.5	− 1.0	− 3.6	− 1.1
3 4	− 2.2	− 3.0	− 2.0	− 7.2	− 2.2
5 6	− 1.1	− 1.5	− 1.0	− 3.6	− 1.1
7 8	− 4.3	− 6.1	− 4.1	−14.5	− 4.3
9 10	− 1.1	− 1.5	− 1.0	− 3.6	− 1.1
11 12	− 2.2	− 3.0	− 2.0	− 7.2	− 2.2
13 14	− 1.1	− 1.5	− 1.0	− 3.6	− 1.1
2 3	+ 1.6	+ 2.2	+ 1.5	+ 5.3	+ 1.6
4 5	+ 1.6	+ 2.2	+ 1.5	+ 5.3	+ 1.6
4 7	+ 3.3	+ 4.5	+ 2.9	+10.7	+ 3.3
6 7	+ 4.9	+ 6.7	+ 4.4	+16.0	+ 4.9
8 9	+ 4.9	+ 6.7	+ 4.4	+16.0	+ 4.9
8 11	+ 3.3	+ 4.5	+ 2.9	+10.7	+ 3.3
8 15	+ 6.3	+ 9.0	+ 6.0	+21.3	+ 6.3
10 11	+ 1.6	+ 2.2	+ 1.5	+ 5.3	+ 1.6
12 15	+ 9.6	+13.5	+ 8.9	+32.0	+ 9.6
12 13	+ 1.6	+ 2.2	+ 1.5	+ 5.3	+ 1.6
14 15	+11.2	+15.6	+10.4	+37.2	+11.2
15 15′	+ 0.9	+ 1.0	+ 0.5	+ 2.4	+ 0.9

EXAMPLE 6. MODIFIED FINK TRUSS WITH 10 PANELS

This modified type in Fig. 80 may be used when the number of panels is required to be 10 instead of 8, 16, etc.

121. Programme.—Truss of type in Fig. 80: span, 100 ft.; rise of upper chord, 20 ft.; rise of lower chord, 1.5 ft.; 10 panels; materials, steel truss and longleaf pine rafters and purlins; covering of painted tin on 7/8-inch sheathing, 2-inch rafters; wooden purlins; trusses 15 ft. on centres; location at Omaha, Neb.; latitude about 41° north; medium exposure; no rolls.

122. Dimensions.—

$$\text{Tan}\, i = \frac{20}{50} = 0.4000 = \tan 21.8°.$$

$$l = \frac{100}{10} = 10.00 \text{ ft.} \qquad l' = \frac{l}{\cos 21.8°} = 10.77 \text{ ft.}$$

$$A = 10.77 \times 15.00 = 161.55 \text{ sq. ft.} = \text{apex area.}$$

123. Apex Loads.—

$$\text{Truss} = \frac{100}{25} + \frac{100^2}{12600} = 4.794 \text{ lbs. per horizontal sq. ft.}$$

$$\text{Snow} = 2.5\,(41° - 35°) = 15.00 \text{ lbs. per horizontal sq. ft.}$$

$$\text{Wind} = \frac{8}{9} \times 21.8° = 19.36 \text{ lbs. per inclined sq. ft.}$$

$P = 161.55\,(2 + 4 + 4 + 3 + 4.791 \cos 21.8°) = 2819$ lbs. $=$ 1.410 tons.

$S = 161.55\,(15.00 \cos 21.8°) = 2250$ lbs. $= 1.125$ tons.

$W = 161.55 \times 19.36 = 3128$ lbs. $= 1.564$ tons.

124. Total Loads on Half Truss.—

Permanent load $= 1.410 \times 4\frac{1}{2} = 6.35$ tons.

Snow load $= 1.125 \times 4\frac{1}{2} = 5.07$ tons.

Wind load $= 1.564 \times 4\frac{1}{2} = 7.04$ tons.

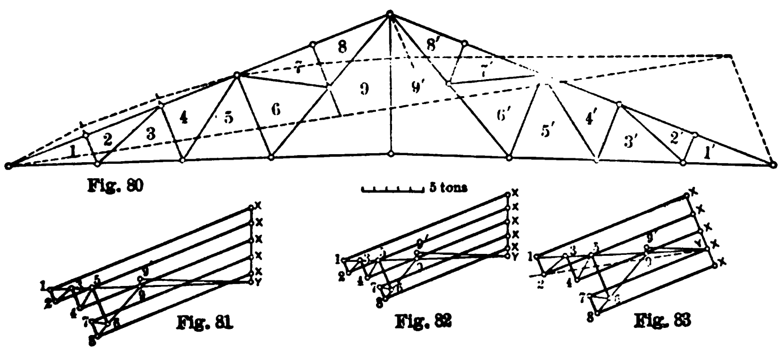
125. Stress Diagrams.—The permanent, snow, and wind stress diagrams are shown in Figs. 81, 82, and 83. No difficulty occurs until the point 6 is to be located. It is evident that it must lie on the stress line 5 6 midway between the stress lines x 7 and x 8, to make the stress lines 6 7 and 6 8 equal, and also that the points 8, 6, and 9 may lie on a straight line parallel to the members 6 7 and 8 9. Since the lower chord is cambered, there will be a tensile stress in the middle vertical 9 9′. The snow stresses may be computed from the permanent stresses in the manner previously indicated.

126. Stress Sheet for Example 6.—

Member.	*P*-stress.	*S*-stress.	*W*-stress.	Maximum.	Minimum.
X 1	−18.7	−14.9	−13.1	−46.7	−18.7
X 2	−18.2	−14.5	−13.1	−45.8	−18.2
X 4	−15.9	−12.6	−10.9	−39.4	−15.9
X 7	−14.7	−11.6	−10.2	−36.5	−14.7
X 8	−14.2	−11.2	−10.2	−35.6	−14.2
Y 1	+17.4	+13.9	+13.9	+45.2	+17.4
Y 3	+15.5	+12.3	+11.7	+39.5	+15.5
Y 5	+13.6	+10.8	+ 9.3	+33.7	+13.6
Y 9	+ 9.7	+ 7.8	+ 4.8	+22.3	+ 9.7
1 2	− 1.3	− 1.1	− 1.5	− 3.9	− 1.3
3 4	− 2.0	− 1.6	− 2.4	− 6.0	− 2.0
5 6	− 3.3	− 2.6	− 3.9	− 9.8	− 3.3
7 8	− 1.3	− 1.1	− 1.5	− 3.9	− 1.3
2 3	+ 1.9	+ 1.6	+ 2.3	+ 5.8	+ 1.9
4 5	+ 2.3	+ 1.8	+ 2.7	+ 6.8	+ 2.3
6 7	+ 1.4	+ 1.1	+ 1.5	+ 4.0	+ 1.4
6 9	+ 4.2	+ 3.3	+ 2.4	+ 9.9	+ 4.2
8 9	+ 5.6	+ 4.3	+ 6.5	+16.4	+ 5.6
9 9′	+ 0.6	+ 0.5	+ 0.3	+ 1.4	+ 0.6

Example 7.—Unsymmetrical Fink Truss

127. Description.—A roof is sometimes required to have slopes of equal or unequal inclination, but of different lengths and resting



Figs. 80–83.—Fink Truss with Ten Panels; Cambered.

on walls of different heights, as in Fig. 84. It then becomes necessary to draw complete permanent and snow stress diagrams, as well as

complete wind stress diagrams for the wind acting on each side of the roof. Owing to the large span in this case, the rolls must be tried at both windward and leeward ends of the truss.

128. Programme.—Type of truss as in Fig. 84: span, 150 ft.; rise 20 ft. above higher and 40 ft. above lower end centres of truss, making 20 ft. difference in the height of the two ends and the walls; ridge, 100 ft. distant horizontally from lower end and 50 ft. from higher end centres of truss; slopes of equal inclination; material, steel; covering of painted tin on 7/8-inch longleaf pine sheathing; steel rafters and purlins; steel trusses with rolls under one end; trusses, 20 ft. on centres; location at Urbana, Ill., latitude about 40 1/8° north; medium exposure.

129. Dimensions.—

$$\text{Tan } i = \frac{40}{100} = 0.4000 = \tan 21.8°.$$

$$l = \frac{150}{12} = 12.50 \text{ ft.} \qquad l' = \frac{12.50}{\cos 21.8°} = 13.46 \text{ ft.}$$

$$A = 13.46 \times 20.00 = 269.20 \text{ sq. ft.} = \text{apex area.}$$

130. Apex Loads.—

$$\text{Truss} = \frac{150}{25} + \frac{150^2}{12600} = 7.79 \text{ lbs. per horizontal sq. ft.}$$

$$\text{Snow} = 2.5\,(40.\,13° - 35°) = 12.81 \text{ lbs. per horizontal sq. ft.}$$

$$\text{Wind} = \frac{8}{9} \times 40.\,13° = 19.36 \text{ lbs. per inclined sq. ft.}$$

$$P = 269.20\,(2 + 4 + 3 + 3 + 7.79 \cos 21.8°) = 5180 \text{ lbs.} = 2.590 \text{ tons.}$$

$$S = 269.20\,(12.81 \cos 21.8°) = 3201 \text{ lbs.} = 1.602 \text{ tons.}$$

$$W = 269.20 \times 19.36 = 5212 \text{ lbs.} = 2.606 \text{ tons.}$$

131. Total Loads on Entire Truss.—

Permanent load $= 2.590 \times 11 = 28.49$ tons.

Snow load $= 1.602 \times 11 = 17.62$ tons.

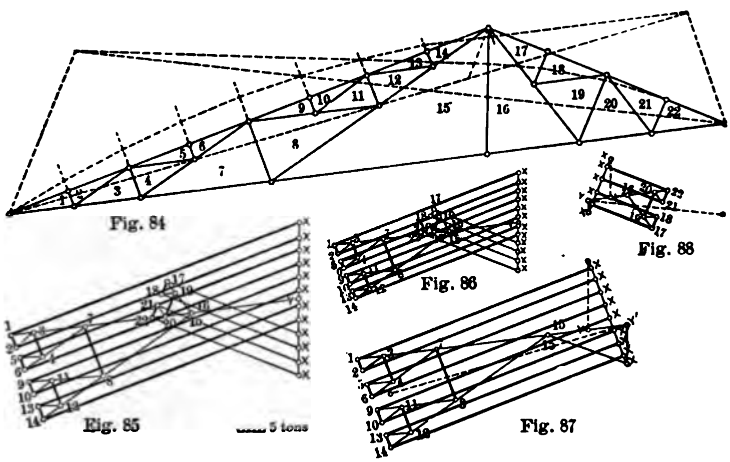
Wind load $= 2.606 \times 7\ 1/2 = 19.55$ tons on long slope.

Wind load $= 2.606 \times 3\ 1/2 = 9.12$ tons on short slope.

These wind loads act alternately on the two slopes of the roof and not at the same time.

132. Stress Diagrams.—The permanent stress diagram is drawn in Fig. 85 for the entire truss, since the stresses in the left and right members are unequal in this case. The snow stress diagram is similar in form in Fig. 86, or the stresses may be obtained by the proportion : 2.590 : 1.602 : : permanent stress : snow stress.

133. Two Wind Stress Diagrams Required.—Since the wind may act on either slope and the expansion rolls may be placed under



Figs. 84–88.—Fink Truss with Unequal Sides.

either end of the truss, separate wind stress diagrams are required in Fig. 87 for wind on long slope and in Fig. 88 for it on the short slope. The dividing point in Fig. 87 is found as before, and the line $y\,y'$ is then drawn through it parallel to the lower chord; the load line is extended upward by the half load at left end; verticals are then drawn through ends of load line to intersect the inclined line at y and y'. The wind stress diagram is then easily completed. The diagram in Fig. 88 is drawn in the same manner for the wind acting on the short slope. The larger wind stresses are measured from the point y' in Fig. 87 and from y in Fig. 88.

134. Stress Sheet for Example 7.—

Member.	*P*-stress.	*S*-stress.	*W*-stress.	Maximum.	Minimum.
X 1	−57.9	−35.6	−47.8	−141.3	−57.9
X 2	−56.9	−35.0	−47.8	−139.7	−56.9
X 5	−56.0	−34.4	−47.8	−138.2	−56.0
X 6	−55.0	−33.8	−47.8	−136.6	−55.0
X 9	−54.0	−33.2	−47.8	−135.0	−54.0
X 10	−54.0	−32.6	−47.8	−134.4	−54.0
X 13	−54.0	−32.0	−47.8	−133.8	−54.0
X 14	−54.0	−31.5	−47.8	−133.3	−54.0
X 17	−26.6	−16.3	−15.3	− 58.2	−26.6
X 18	−27.6	−16.9	−15.3	− 59.8	−27.6
X 21	−28.6	−17.5	−15.3	− 61.4	−28.6
X 22	−29.5	−18.0	−15.3	− 62.8	−29.5
Y 1	+54.2	+33.3	+52.1	+139.6	+54.2
Y 3	+49.4	+30.3	+46.8	+126.5	+49.4
Y 7	+39.8	+24.3	+36.0	+100.1	+39.8
Y 15	+20.1	+12.3	+14.4	+ 46.8	+20.1
Y 20	+24.9	+15.4	+14.4	+ 54.7	+24.9
Y 22	+27.6	+16.9	+15.4	+ 59.9	+27.6
1 2	− 2.4	− 1.5	− 2.7	− 6.6	− 2.4
3 4	− 4.8	− 3.0	− 5.3	− 13.1	− 4.8
5 6	− 2.4	− 1.5	− 2.7	− 6.6	− 2.4
7 8	− 9.5	− 6.0	−10.6	− 26.1	− 9.5
9 10	− 2.4	− 1.5	− 2.7	− 6.6	− 2.4
11 12	− 4.8	− 3.0	− 5.3	− 13.1	− 4.8
13 14	− 2.4	− 1.5	− 2.7	− 6.6	− 2.4
17 18	− 2.4	− 1.5	− 2.7	− 6.6	− 2.4
19 20	− 4.8	− 3.0	− 5.3	− 13.1	− 4.8
21 22	− 2.4	− 1.5	− 2.7	− 6.6	− 2.4
2 3	+ 4.8	+ 3.0	+ 5.3	+ 13.1	+ 4.8
4 5	+ 4.8	+ 3.0	+ 5.3	+ 13.1	+ 4.8
4 7	+ 9.7	+ 6.0	+10.8	+ 26.1	+ 9.7
6 7	+14.6	+ 9.0	+16.1	+ 39.7	+14.6
8 9	+14.6	+ 9.0	+16.1	+ 39.7	+14.6
8 11	+ 9.7	+ 6.0	+10.8	+ 26.5	+ 9.7
8 15	+19.6	+12.0	+21.7	+ 53.3	+19.6
10 11	+ 4.8	+ 3.0	+ 5.3	+ 13.1	+ 4.8
12 15	+29.4	+18.0	+32.6	+ 80.0	+29.4
14 15	+34.5	+21.2	+37.8	+ 93.5	+34.6
12 13	+ 4.8	+ 3.0	+ 5.3	+ 13.1	+ 4.8
15 16	+ 0.0	+ 0.0	+ 0.0	+ 0.0	+ 0.0
16 17	+ 7.5	+ 4.6	+ 8.0	+ 20.1	+ 7.5
16 19	+ 5.0	+ 3.1	+ 2.5	+ 10.6	+ 5.0
18 19	+ 2.6	+ 1.5	+ 2.6	+ 6.7	+ 2.6
20 21	+ 2.6	+ 1.5	+ 2.6	+ 6.7	+ 2.6

Example 8.—A Mansard Truss

135. Description.—Each side of a mansard roof is here composed of two slopes of different inclinations, the steeper one sometimes being slated or tiled, the upper one or deck being covered with tin or felt and gravel.

136. Programme.—Truss of type in Fig. 89: span, 100 ft.; rise, 20 ft. at ridge and 16 ft. at edge of deck roof; 10 panels; materials, steel truss and longleaf pine lumber; covering of painted tin on 7/8-inch sheathing; wooden rafters and purlins; trusses, 15 ft. on centres; location at Boston, about 42½° north latitude; medium exposure.

137. Dimensions.—

Let i' = inclination of side slope; i'' = inclination of deck slope.

$$\text{Tan } i' = \frac{16}{20} = 0.8000 = \tan 38.7°; \tan i'' = \frac{4}{30} = 0.1333 = \tan 7.6°.$$

$$l' = \frac{10.00}{\cos 38.7°} = 12.81 \text{ ft.}; \quad l'' = \frac{10.00}{\cos 7.6°} = 10.09 \text{ ft.}$$

$$A' = 12.81 \times 15.00 = 192.15 \text{ sq. ft.} = \text{side apex area.}$$

$$A'' = 10.09 \times 15.00 = 151.35 \text{ sq. ft.} = \text{deck apex area.}$$

138. Apex Loads.—

Truss = 4.794 lbs. per horizontal sq. ft., as before.

Snow = 2.5 (42.5°) = 18.75 lbs. per horizontal sq. ft.

$$\text{Wind on side} = \frac{8}{9} \times 38.7° = 34.40 \text{ lbs. per inclined sq. ft.}$$

$$\text{Wind on deck} = \frac{8}{9} \times 7.6° = 6.76 \text{ lbs. per inclined sq. ft.}$$

Loads for side.

P = 192.15 (2 + 4 + 3 + 3 + 4.794 cos 38.7°) = 2640 lbs. = 1.320 tons.

S = 192.15 (18.75 cos 38.7°) = 2809 lbs. = 1.405 tons.

W = 192.15 × 34.40 = 6610 lbs. = 3.305 tons.

Loads for deck.

P = 151.35 (2 + 4 + 3 + 3 + 4.794 cos 7.6°) = 2384 lbs. = 1.192 tons.

S = 151.35 (18.75 cos 7.6°) = 2827 lbs. = 1.413 tons.

W = 151.35 × 6.76 = 1023 lbs. = 0.517 ton.

139. Total Loads on Half Truss.—

Permanent loads $= 1.320 \times 1\frac{1}{2} + 1.192 \times 3 = 5.56$ tons.

Snow loads $= 1.405 \times 1\frac{1}{2} + 1.413 \times 3 = 6.35$ tons.

Wind loads on side $= 3.305 \times 1\frac{1}{2} = 4.96$ tons.

Wind loads on deck $= 0.517 \times 3 = 1.55$ tons.

140. Stress Diagrams.—Since the apex permanent loads are larger for the sides than for the deck, and the apex snow loads are practically equal for both side and deck, separate stress diagrams must be drawn for permanent, snow, and wind loads. The nature of the stresses in the web members is found to differ beneath sides and deck, as apparent from the stress diagrams and stress sheet. The

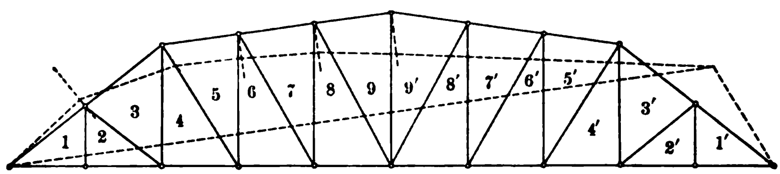


Fig. 89

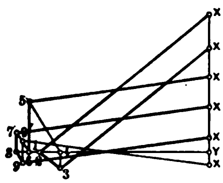


Fig. 90

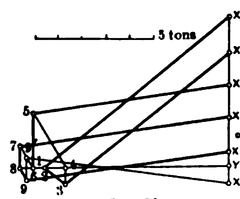


Fig. 91

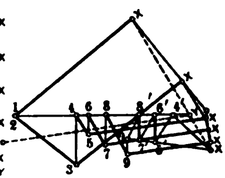


Fig. 92

FIGS. 89–92.—Mansard Roof Truss.

stresses in 7 8 and 6 7 are also here found to reverse under the wind pressure. Hence this truss should be constructed of steel, which will also produce the best appearance, if it be visible in the interior of the building.

141. Small Stresses in Web Members.—The stresses in the web members are found to be very small, since this truss approximates to a parabola in form, with vertex at the ridge; if the apexes of a parabolic truss are equidistant horizontally and are equally loaded, stresses occur in the chords alone, not in the web members. Hence the small stresses in the web members in this case.

Half the permanent stress diagram is given in Fig. 90; half the snow stress diagram in Fig. 91; the entire wind stress diagram is in Fig. 92.

142. Stress Sheet for Example 8.—

Member.	*P*-stress.	*S*-stress.	*W*-stress.	Maximum.	Minimum.
X 1	− 8.9	−10.2	− 6.4	−25.5	− 8.9
X 3	− 7.1	− 9.1	− 5.6	−22.6	− 7.1
X 5	− 7.5	− 8.7	− 5.0	−21.2	− 7.5
X 7	− 7.7	− 9.2	− 4.4	−21.5	− 7.7
X 9	− 7.7	− 9.0	− 3.6	−20.3	− 7.7
Y 1	+ 6.9	+ 7.9	+ 7.4	+22.2	+ 6.9
Y 2	+ 6.9	+ 7.9	+ 7.4	+22.2	+ 6.9
Y 4	+ 6.1	+ 7.1	+ 4.7	+17.9	+ 6.1
Y 6	+ 7.4	+ 8.6	+ 4.3	+20.3	+ 7.4
Y 8	+ 7.9	+ 9.2	+ 3.6	+20.7	+ 7.9
1 2	+ 0.0	+ 0.0	+ 0.0	+ 0.0	+ 0.0
3 4	+ 0.7	+ 0.7	+ 2.1	+ 3.5	+ 0.7
5 6	− 2.0	− 2.4	+ 0.8	− 4.4	− 1.2
7 8	− 0.8	− 1.0	+ 1.2	− 1.8	+ 0.4
9 9	+ 0.8	+ 1.0	+ 0.7	+ 2.5	+ 0.8
2 3	− 1.1	− 1.2	− 3.4	− 5.7	− 1.1
4 5	+ 2.4	+ 2.9	− 0.9	+ 5.3	+ 1.5
6 7	+ 0.9	+ 1.1	− 1.4	+ 2.2	− 0.5
8 9	− 0.5	− 0.5	− 1.8	− 2.8	− 0.5

Members 7 8 and 6 7 must be arranged to safely resist each of the two different kinds of stress found in them.

Example 9.—Segmental Crescent Truss

143. Description.—The external surface of the roof is here cylindrical, and its inclination regularly diminishes from either side to the top, at which it becomes horizontal. The intensity of the wind pressure varies in the same manner, making it rather difficult to accurately obtain the magnitude and direction of the wind load supported at any particular apex of the truss. An approximate method is sufficiently accurate for practice and is more easily applied.

144. Average Inclination at Apex.—In Fig. 93, bisect the curve of the upper chord between apexes *A*, *B*, *C*, *D*, *E*, and *F*; draw a horizontal and a vertical through each bisecting point, and draw chord *a c*. Then the angle of inclination *c a b* of the chord *a c* may be safely assumed to be the average inclination of the apex area supported at apex *B*.

Then $\tan i_b^\circ = \dfrac{b\,c}{a\,b}$; $l' = \dfrac{l}{\cos i} = a\,c$; $A = d \times l'$; for B.

145. Members Straight or Curved between Apexes.—The members $A\ B$, etc., of the upper chord may be curved between adjacent apexes, but are then weaker and require greater cross-sections than if straight, or they may be straight between apexes and the wooden rafters be cut to the required curve to receive the sheathing. The rafters are frequently omitted in curved roofs, when intermediate purlins are inserted to receive the sheathing, bent to the curve of the surface. Then the intermediate purlins are easily blocked out to the curve of the sheathing. This truss is to be constructed of straight members, but the apexes are required to lie in the curve. It is divided into panels by verticals equidistant horizontally.

146. Programme.—Truss of type in Fig. 94: span, 100 ft.; rise of upper chord, 25 ft.; of lower chord, 10 ft.; materials. steel trusses,

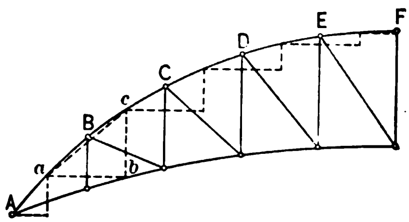


FIG. 93.—Inclinations at Apexes.

longleaf pine lumber; covering of painted tin on 7/8-inch sheathing; no rafters; longleaf pine purlins, spaced as necessary and blocked out to curve; trusses, 15 ft. on centres; location at Philadelphia, latitude about 41° north; medium exposure.

147. Dimensions.—Bisect curve of upper chord midway between apexes and draw horizontals and verticals through bisecting points; measure horizontal and vertical subtending each apex. Letter the successive apexes A, B, C, D, E, and F.

$$\text{Tan } i_b^\circ = \frac{8.43}{10} = 0.8430 = \tan 40.1^\circ, \text{ inclination at } B.$$

$$\text{Tan } i_c^\circ = \frac{5.49}{10} = 0.5490 = \tan 28.8^\circ, \text{ inclination at } C.$$

$$\text{Tan } i_d^\circ = \frac{3.41}{10} = 0.3410 = \tan 18.8^\circ, \text{ inclination at } D.$$

$$\text{Tan } i_e^\circ = \frac{1.67}{10} = 0.1670 = \tan 9.5^\circ, \text{ inclination at } E.$$

$$\text{Tan } i_f^\circ = 0.0000 = \tan 0.0^\circ, \text{ inclination at } F. \text{ (Horizontal.)}$$

$l_b' = \dfrac{10}{\cos 40.1^\circ} = 13.08$ ft.; $A_b = 13.08 \times 15.00 = 196.2$ sq. ft. at B.

$l_c' = \dfrac{10}{\cos 28.8^\circ} = 11.41$ ft.; $A_c = 11.41 \times 15.00 = 171.2$ sq. ft. at C.

$l_d' = \dfrac{10}{\cos 18.8^\circ} = 10.56$ ft.; $A_d = 10.56 \times 15.00 = 158.4$ sq. ft. at D.

$l_e' = \dfrac{10}{\cos 9.5^\circ} = 10.14$ ft.; $A_e = 10.14 \times 15.00 = 152.1$ sq. ft. at E.

$l_f' = \dfrac{10}{\cos 0.0^\circ} = 10.00$ ft.; $A_f = 10.00 \times 15.00 = 150.0$ sq. ft. at F.

148. Apex Loads.—Truss, as before, 4.794 lbs. per horizontal sq. ft.
Snow $= 2.5\,(41^\circ - 35^\circ) = 15.00$ lbs. per horizontal sq. ft.
$\text{Snow}_b = 15.0 \cos 40.1^\circ = 11.47$ lbs. per inclined sq. ft. at B.
$\text{Snow}_c = 15.0 \cos 28.8^\circ = 13.14$ lbs. per inclined sq. ft. at C.
$\text{Snow}_d = 15.0 \cos 18.8^\circ = 14.19$ lbs. per inclined sq. ft. at D.
$\text{Snow}_e = 15.0 \cos 9.5^\circ = 14.79$ lbs. per inclined sq. ft at E.
$\text{Snow}_f = 15.0 \cos 0.0^\circ = 15.00$ lbs. per inclined sq. ft. at F.

Wind $= \dfrac{8}{9}\, i^\circ$ in lbs. per inclined sq. ft. of apex area.

$\text{Wind}_b = \dfrac{8}{9} \times 40.1^\circ = 35.63$ lbs. per inclined sq. ft. at B.

$\text{Wind}_c = \dfrac{8}{9} \times 28.8^\circ = 25.60$ lbs. per inclined sq. ft. at C.

$\text{Wind}_d = \dfrac{8}{9} \times 18.8^\circ = 16.71$ lbs. per inclined sq. ft. at D.

$\text{Wind}_e = \dfrac{8}{9} \times 9.5^\circ = 8.44$ lbs. per inclined sq. ft. at E.

$\text{Wind}_f = \quad = 0.00$ lbs. per inclined sq. ft. at F.

$P_b = 196.2\,(2 + 4 + 0 + 4 + 4.794 \cos 40.1^\circ) = 2682$ lbs. $=$ 1.341 T. at B.

$P_c = 171.2\,(2 + 4 + 0 + 4 + 4.794 \cos 28.8^\circ) = 2431$ lbs. $=$ 1.216 T. at C.

$P_d = 158.4\,(2 + 4 + 0 + 4 + 4.794 \cos 18.8^\circ) = 2303$ lbs. $=$ 1.152 T. at D.

$P_e = 152.1\,(2 + 4 + 0 + 4 + 4.794 \cos 9.5^\circ) = 2240$ lbs. $=$ 1.120 T. at E.

$P_f = 150.0\,(2 + 4 + 0 + 4 + 4.794 \cos 0.0^\circ) = 2219$ lbs. $=$ 1.110 T. at F.

$S_b = 196.2\,(15.0 \cos 40.1^\circ) = 2250$ lbs. $= 1.125$ tons at B.
$S_c = 171.2\,(15.0 \cos 28.8^\circ) = 2250$ lbs. $= 1.125$ tons at C.

$S_d = 158.4\ (15.0 \cos 18.8°) = 2250$ lbs. $= 1.125$ tons at D.
$S_e = 152.1\ (15.0 \cos 9.5°) = 2250$ lbs. $= 1.125$ tons at E.
$S_f = 150.0\ (15.0 \cos 0.0°) = 2250$ lbs. $= 1.125$ tons at F.
$W_b = 196.2 \times 35.63 = 6991$ lbs. $= 3.496$ tons at B.
$W_c = 171.2 \times 25.60 = 4383$ lbs. $= 2.192$ tons at C.
$W_d = 158.4 \times 16.71 = 2647$ lbs. $= 1.324$ tons at D.
$W_e = 152.1 \times 8.44 = 1284$ lbs. $= 0.642$ tons at E.
$W_f = 150.0 \times 0.00 = 0$ lbs. $= 0.000$ tons at F.

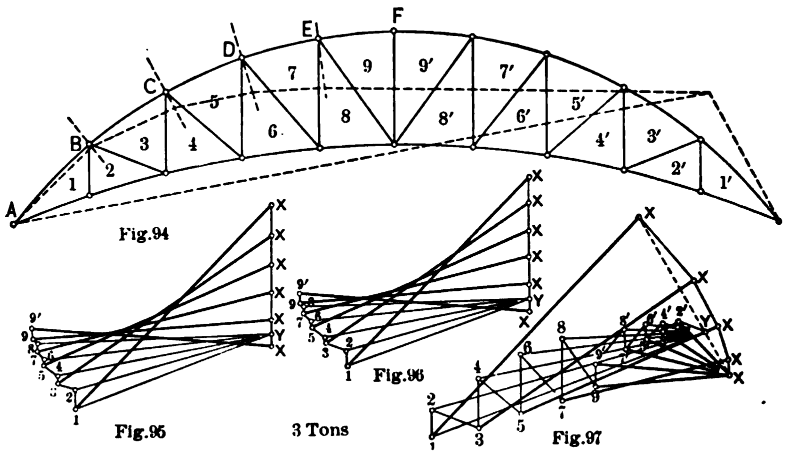
149. Total Loads on Half Truss.—

Permanent load $= 1.341 + 1.216 + 1.152 + 1.120 + \frac{1.110}{2} =$ 5.38 tons.

Snow load $= 1.125 \times 4\frac{1}{2} = 5.06$ tons.

Wind load must be found graphically.

150. Stress Diagram for Example 9.—A stress line must be drawn parallel to the chord joining the apexes at the ends of the



FIGS. 94–97.—Segmental Crescent Truss.

corresponding member, when this is curved. Stresses in web members are quite small, excepting for wind loads, and their nature may then be reversed. The dimensions of such a member are then to be made sufficient to safely resist both kinds of stress, or a counter diagonal is to be inserted in the panel to resist the reversed stress. Permanent and snow stress diagrams must both be drawn.

151. Stress Sheet for Example 9.—

Member.	*P*-stress.	*S*-stress.	*W*-stress *W*.	*W*-stress *L*.	Maximum.	Minimum.
X 1	−11.5	−10.7	−12.1	− 3.1	−34.5	−11.5
X 3	−10.5	−10.0	−10.5	− 3.3	−31.0	−10.5
X 5	−10.1	− 9.7	− 8.7	− 3.8	−28.5	−10.1
X 7	− 9.9	− 9.5	− 7.0	− 4.5	−26.4	− 9.9
X 9	− 9.7	− 9.3	− 5.4	− 5.4	−24.4	− 9.7
Y 1	+ 8.4	+ 7.8	+11.8	+ 1.1	+28.0	+ 8.4
Y 2	+ 8.2	+ 7.6	+11.5	+ 1.1	+27.3	+ 8.2
Y 4	+ 8.9	+ 8.4	+ 9.3	+ 1.7	+26.6	+ 8.9
Y 6	+ 9.3	+ 8.9	+ 7.4	+ 2.4	+25.6	+ 9.3
Y 8	+ 9.6	+ 9.2	+ 5.8	+ 3.3	+24.6	+ 9.6
1 2	+ 0.7	+ 0.7	+ 1.0	+ 0.1	+ 2.4	+ 0.7
3 4	+ 0.2	+ 0.1	+ 2.1	− 0.3	+ 2.4	− 0.1
5 6	+ 0.2	+ 0.1	+ 2.5	− 0.6	+ 2.8	− 0.4
7 8	+ 0.3	+ 0.3	+ 2.6	+ 0.9	+ 3.2	− 0.6
9 9′	+ 0.5	+ 0.3	+ 0.9	− 0.9	+ 1.7	+ 0.5
2 3	+ 0.9	+ 1.0	− 2.1	+ 0.7	+ 2.6	− 1.2
4 5	+ 0.8	+ 0.8	− 2.2	+ 1.0	+ 2.6	− 1.4
6 7	+ 0.5	+ 0.5	− 2.6	+ 1.4	+ 2.4	− 2.1
8 9	+ 0.2	+ 0.5	− 2.5	+ 1.8	+ 2.2	− 2.3

Example 10.—Semicircular Crescent Truss

152. Description.—The external surface of this roof is a semicircular cylinder supported by crescent trusses, each of which is divided into panels by equidistant radii drawn from the centre of the upper chord. The members forming the chords may be either curved or straight, then being the chord of the arc of each panel of the chord. Each panel of the upper chord is bisected, as in Example 9, a vertical and a horizontal then being drawn through each bisecting point in order to obtain the average inclination of the roof for the corresponding apex and the values of l' in the manner explained for Example 9. Or the inclinations may here be found by the following formulas.

153. Inclinations by Formulas.—

Let β° = angle of inclination of radial drawn to an apex.
m = number of panels between apex and nearest end of truss.
n = number of panels in the half truss.

Then $\frac{90°}{n}$ = angle at centre subtended by one panel of chord. (34)

$i° = 90° - m\frac{90°}{n}$ = average inclination at apex considered. (35)

Let D = diameter of semicircular upper chord in ft.

Then $\frac{\pi D}{4n}$ = panel length measured on the curve, which is slightly greater than if computed as in the last example. . . . . (36)

But it would be preferable to construct this truss entirely of straight chord members connecting the apexes.

154. Programme.—Type of truss as in Fig. 98: span, 100 ft.; rise of upper chord, 50 ft.; rise of lower chord, 37.5 ft.; depth of truss, 12.5 ft. at centre; 14 equal panels separated by radials drawn to upper chord; materials, steel trusses, Washington fir lumber; covering of painted tin on 7/8-inch wooden sheathing; no rafters; wooden purlins, blocked out to curve; trusses, 16 ft. on centres; location at Kansas City, Mo.; latitude, about 39° north; medium exposure.

155. Dimensions.—Bisect each panel on curve of upper chord; draw vertical and horizontal through each bisecting point and measure them, in order to compute the values of i and l' at the corresponding apexes as before.

$$\text{Tan } i_b = \frac{10.95}{2.48} = 4.4153 = \tan 77.2°, \text{ inclination at } B.$$

$$\text{Tan } i_c = \frac{10.17}{4.87} = 2.0883 = \tan 64.4°, \text{ inclination at } C.$$

$$\text{Tan } i_d = \frac{8.78}{6.95} = 1.2633 = \tan 51.6°, \text{ at } D.$$

$$\text{Tan } i_e = \frac{7.00}{8.73} = 0.8018 = \tan 38.7°, \text{ at } E.$$

$$\text{Tan } i_f = \frac{4.90}{10.12} = 0.4842 = \tan 25.8°, \text{ at } F.$$

$$\text{Tan } i_g = \frac{2.54}{10.97} = 0.2315 = \tan 13.0°, \text{ at } G.$$

$$\text{Tan } i_h = \qquad = 0.0000 = \tan 0.0°, \text{ at } H.$$

$$l' = \frac{100 \times \pi}{2 \times 14} = 11.22 \text{ ft.}$$

A = 11.22 × 16.00 = 179.52 sq. ft. = each apex area.

156. Apex Loads.—Truss = 4.794 lbs. per horizontal sq. ft., as before.

It will here be best to average the weight of the truss per sq. ft. of the surface of the roof.

Then $4.794 \times 100 \times 16.00 = 7670$ lbs. = total weight of the truss.

And $\dfrac{7670 \times 2}{100 \times 16 \times \pi} = 3.051$ lbs. per sq. ft. of roof surface.

Snow = 2.5 (39° − 35°) = 10.00 lbs. per horizontal sq. ft.

Wind $= \dfrac{8}{9} \times i$ = wind pressure per inclined sq. ft., varying with i.

$P = 179.52\,(2 + 4 + 0 + 4 + 3.05) = 2343$ lbs. = 1.172 tons per apex.
$S_b = 179.52\,(10.00 \cos 77.3°) = 395$ lbs. = 0.198 tons at B.
$S_c = 179.52\,(10.00 \cos 64.4°) = 776$ lbs. = 0.388 tons at C.
$S_d = 179.52\,(10.00 \cos 51.6°) = 1115$ lbs. = 0.558 tons at D.
$S_e = 179.52\,(10.00 \cos 38.7°) = 1401$ lbs. = 0.701 tons at E.
$S_f = 179.52\,(10.00 \cos 25.8°) = 1616$ lbs. = 0.808 tons at F.
$S_g = 179.52\,(10.00 \cos 13.0°) = 1749$ lbs. = 0.875 tons at G.
$S_h = 179.52 \times 10.00 \times 1.00 = 1795$ lbs. = 0.898 tons at H.
$W_b = 179.52 \times 40.00 = 7181$ lbs. = 3.591 tons at B.
$W_c = 179.52 \times 40.00 = 7181$ lbs. = 3.591 tons at C.
$W_d = 179.52 \times 40.00 = 7181$ lbs. = 3.591 tons at D.

$W_e = 179.52 \times \dfrac{8}{9} \times 38.7° = 6175$ lbs. = 3.088 tons at E.

$W_f = 179.52 \times \dfrac{8}{9} \times 25.8° = 4116$ lbs. = 2.058 tons at F.

$W_g = 179.52 \times \dfrac{8}{9} \times 13.0° = 2073$ lbs. = 1.037 tons at G.

$W_h = 179.52 \times 0.0 = 0$ lbs. = 0.000 tons at H.

157. Total Loads on Half Truss.—

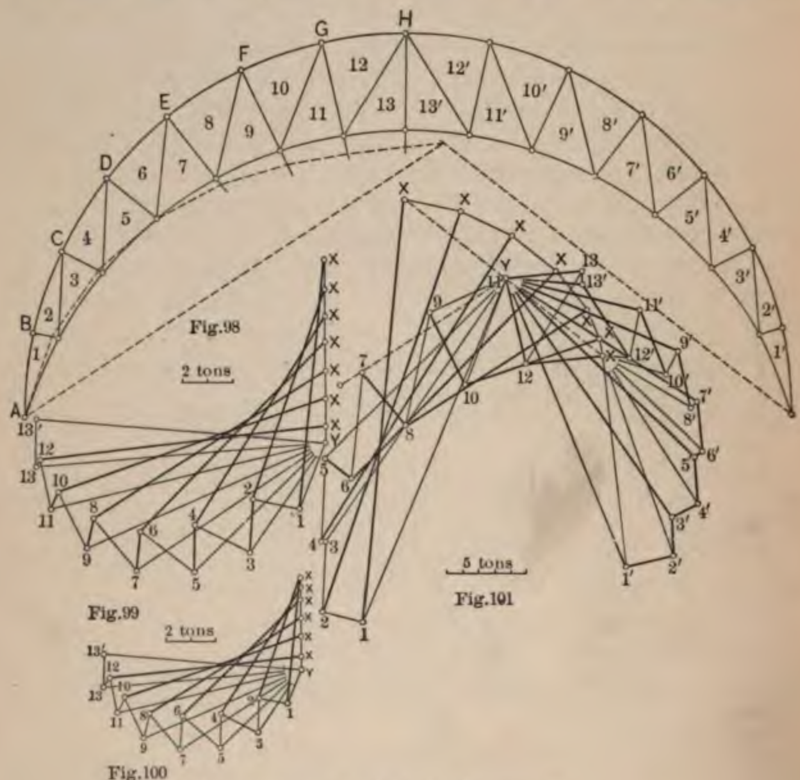
Permanent load = $1.172 \times 6\frac{1}{2} = 7.62$ tons.

Snow load $= 0.198 + 0.388 + 0.558 + 0.701 + 0.808 + 0.875 + \dfrac{0.898}{2} = 3.98$ tons.

Wind load must be obtained graphically, each apex load acting radially.

158. Stress Diagrams.—The permanent, snow, and wind stress diagrams are drawn in Figs. 99, 100, and 101. Stresses in members of upper and lower chords are here reversed on the leeward side of the truss. Or in Fig. 101, all y-stress lines lying on the right of a vertical through the y-point are in compression; also all x-stress lines lying on the right of a vertical through the lower end of the load line are in tension.

159. Reversed Stresses.—It is evident from the stress diagrams and the stress sheet, that some members have reversed stresses, *i.e.*, that the nature of the stresses in them are changed. When such reversed stresses occur in members of upper or lower chords, such members must be dimensioned to safely resist both kinds of stresses. The same may be applied to web members, or the other diagonal



Figs. 98–101.—Semicircular Crescent Truss.

of the trapezoidal panel may be inserted, as suggested in Example 2. This usually prevents the reversal of the stress. But only a single diagonal in each panel can be used in drawing the stress diagram, since the case would become indeterminate otherwise. The diagonal and counter diagonal can then be dimensioned for the stresses acting in each. But it is usually more economical to use only the regular diagonals of the panels, omitting counter diagonals.

160. Stress Sheet for Example 10.—

Member.	*P*-stress.	*S*-stress.	*W*-stress *W*.	*W*-stress *L*.	Maximum.	Minimum.
X 1	−10.5	− 5.5	−28.2	+14.1	−44.2	+ 3.6
X 2	− 9.3	− 5.3	−28.5	+14.2	−43.1	+ 4.9
X 4	−10.5	+ 6.4	−23.7	+11.8	−40.6	+ 1.3
X 6	−11.1	− 7.1	−18.9	+ 9.5	−37.1	− 1.6
X 8	−11.3	− 7.6	−14.2	+ 7.2	−33.1	− 4.1
X 10	−11.6	− 7.9	− 9.4	+ 4.8	−28.9	− 6.8
X 12	−11.8	− 8.1	− 5.0	+ 2.1	−24.9	− 9.7
Y 1	+ 3.0	+ 1.7	+25.1	−20.4	+29.8	−17.4
Y 3	+ 5.7	+ 3.3	+21.2	−19.0	+30.2	−13.3
Y 5	+ 7.9	+ 4.8	+16.6	−17.2	+29.3	− 9.3
Y 7	+ 9.5	+ 6.2	+11.4	−15.1	+27.1	− 5.6
Y 9	+10.8	+ 7.3	+ 5.7	−12.5	+23.8	− 1.7
Y 11	+11.6	+ 7.9	+ 0.1	− 9.2	+19.6	+ 2.4
Y 13	+11.9	+ 8.2	− 5.0	− 5.1	+20.1	+ 1.8
1 2	+ 2.0	+ 1.2	+ 2.9	− 3.2	+ 6.1	− 1.2
3 4	+ 2.7	+ 1.7	+ 0.3	− 1.9	+ 4.7	+ 0.8
5 6	+ 2.9	+ 2.0	− 2.1	− 0.8	+ 4.9	+ 0.8
7 8	+ 2.8	+ 2.0	− 4.0	+ 0.5	+ 5.3	− 1.2
9 10	+ 2.5	+ 1.8	− 5.2	+ 1.7	+ 6.0	− 2.7
11 12	+ 2.2	+ 1.5	− 5.6	+ 3.0	+ 6.7	− 3.4
13 13	+ 1.9	+ 1.3	− 0.8	+ 0.8	+ 3.2	+ 1.1
2 3	− 2.5	− 1.5	+ 5.2	− 2.5	− 6.5	+ 2.7
4 5	− 2.0	− 1.4	+ 5.8	− 2.8	− 6.2	+ 3.8
6 7	− 1.6	− 1.4	+ 6.6	− 3.3	− 6.3	+ 5.0
8 9	− 1.3	− 1.1	+ 7.5	− 3.8	− 6.2	+ 6.2
10 11	− 0.8	− 0.7	+ 7.5	− 4.6	− 6.1	+ 6.7
12 13	− 0.2	− 0.3	+ 7.0	− 5.7	− 6.2	+ 6.8

Example 11.—Hemispherical Crescent-trussed Dome

161. Description.—This dome is of hemispherical form and is supported by 16 complete crescent trusses having a common middle vertical. Each truss is also divided into 16 equal panels by radials to upper chord. Hence an area of approximately trapezoidal form is supported at each apex. Each series of apexes in a horizontal plane is connected together by a ring purlin bent to the proper curve, thus composing a horizontal circular ring. On the purlins are fixed meridian rafters, also bent to the curve of a great circle. To these are fastened the wooden sheathing, kerfed on the under side, if necessary, which receives the covering.

Each panel of the upper chord is bisected as before, a vertical and horizontal being drawn through each bisecting point, as in Examples 9 and 10. (Omitted in Fig. 102 for sake of clearness.)

Fig. 103 represents the plan of the portion of the surface of the dome supported by the half truss *A I*. On it are drawn the half purlins supported at the respective apexes.

162. Apex Areas.—The exact computation of the apex area supported at each apex is a problem of considerable difficulty. But an approximate method will be found sufficiently accurate for practical purposes and far more convenient in application. Measure or compute panel length of upper chord and length of purlin at each apex between centre planes of two adjacent trusses. Then panel length × purlin length approximately equals the apex area supported at the corresponding apex.

163. Programme.—Type of truss as in Fig. 102: span, 100 ft.; rise of upper chord, 50 ft.; of lower chord, 40 ft. making depth of truss, 10 ft. at centre; 16 panels between radials to upper chord; material, steel; covering of painted tin on 7/8-inch longleaf pine sheathing, kerfed on under side, if necessary; steel rafters, purlins, and trusses; located at Boston, latitude about 42½° north; medium exposure.

164. Dimensions.—

$$\text{Tan } i_b = \frac{9.67}{1.92} = 5.0365 = \tan 78.8° \text{ at } B.$$

$$\text{Tan } i_c = \frac{9.10}{3.80} = 2.3947 = \tan 67.3° \text{ at } C.$$

$$\text{Tan } i_d = \frac{8.17}{4.45} = 1.8360 = \tan 61.4° \text{ at } D.$$

$$\text{Tan } i_e = \frac{7.00}{6.90} = 1.0145 = \tan 45.4° \text{ at } E.$$

$$\text{Tan } i_f = \frac{5.49}{8.16} = 0.6728 = \tan 33.9° \text{ at } F.$$

$$\text{Tan } i_g = \frac{3.77}{9.08} = 0.4152 = \tan 22.5° \text{ at } G.$$

$$\text{Tan } i_h = \frac{1.92}{9.60} = 0.2000 = \tan 11.3° \text{ at } H.$$

$$\text{Tan } i_i = \quad = 0.000 = \tan 0.0 \text{ at } I.$$

Length of purlin at B = 9.60 ft.
Length of purlin at C = 9.06 ft.
Length of purlin at D = 8.16 ft.
Length of purlin at E = 6.92 ft.
Length of purlin at F = 5.42 ft.
Length of purlin at G = 3.70 ft.
Length of purlin at H = 1.90 ft.
Length of purlin at I = 0.90 ft.

$$l' = \frac{100 \times \pi}{2 \times 16} = 9.817 \text{ ft.}$$

$A_b = 9.817 \times 9.60 = 94.24$ sq. ft. = apex area at B.
$A_c = 9.817 \times 9.06 = 88.94$ sq. ft. = apex area at C.
$A_d = 9.817 \times 8.16 = 80.11$ sq. ft. = apex area at D.
$A_e = 9.817 \times 6.92 = 63.03$ sq. ft. = apex area at E.
$A_f = 9.817 \times 5.42 = 53.21$ sq. ft. = apex area at F.
$A_g = 9.817 \times 3.70 = 36.32$ sq. ft. = apex area at G.
$A_h = 9.817 \times 1.90 = 18.65$ sq. ft. = apex area at H.
$A_i = 9.817 \times 0.90 = 4.30$ sq. ft. = apex area at I.

165. Apex Loads.—

$$\text{Total weight of truss} = \frac{7854 \times 4.794}{16} = 2353 \text{ lbs.}$$

$$\text{Average for truss per apex of upper chord} = \frac{2353}{17} = 138.4 \text{ lbs.}$$

$P_b = 94.21\ (2 + 4 + 4 + 3) + 138.4 = 1364$ lbs. = 0.682 ton.
$P_c = 88.94\ (2 + 4 + 4 + 3) + 138.4 = 1295$ lbs. = 0.648 ton.
$P_d = 80.11\ (2 + 4 + 4 + 3) + 138.4 = 1180$ lbs. = 0.590 ton.
$P_e = 63.03\ (2 + 4 + 4 + 3) + 138.4 = 958$ lbs. = 0.479 ton.
$P_f = 53.21\ (2 + 4 + 4 + 3) + 138.4 = 830$ lbs. = 0.415 ton.
$P_g = 36.32\ (2 + 4 + 4 + 3) + 138.4 = 611$ lbs. = 0.306 ton.
$P_h = 18.65\ (2 + 4 + 4 + 3) + 138.4 = 381$ lbs. = 0.191 ton.
$P_i = 4.3\ (2 + 4 + 4 + 3) + 138.4 = 194$ lbs. = 0.097 ton.

$S_b = 94.24 \times 18.75 \cos 78.8° = 343$ lbs. = 0.172 ton.
$S_c = 88.94 \times 18.75 \cos 67.3° = 644$ lbs. = 0.322 ton.
$S_d = 80.11 \times 18.75 \cos 61.4° = 719$ lbs. = 0.360 ton.
$S_e = 63.03 \times 18.75 \cos 45.4° = 830$ lbs. = 0.415 ton.
$S_f = 53.21 \times 18.75 \cos 33.9° = 828$ lbs. = 0.414 ton.
$S_g = 36.32 \times 18.75 \cos 22.5° = 629$ lbs. = 0.315 ton.
$S_h = 18.65 \times 18.75 \cos 11.3° = 343$ lbs. = 0.172 ton.
$S_i = 4.30 \times 18.75 \cos 0.0° = 81$ lbs. = 0.041 ton.

$W_b = 94.24 \times 40.00$ lbs. $= 3761$ lbs. $= 1.881$ tons.
$W_c = 88.94 \times 40.00$ lbs. $= 3598$ lbs. $= 1.799$ tons.
$W_d = 80.11 \times 40.00$ lbs. $= 3204$ lbs. $= 1.602$ tons.
$W_e = 63.03 \times 40.00$ lbs. $= 2521$ lbs. $= 1.261$ tons.
$W_f = 53.21 \times 30.13$ lbs. $= 1600$ lbs. $= 0.800$ tons.
$W_g = 36.32 \times 20.00$ lbs. $= 726$ lbs. $= 0.363$ tons.
$W_h = 18.65 \times 10.00$ lbs. $= 187$ lbs. $= 0.094$ tons.
$W_i = 4.3 \times 0.00$ lbs. $= 0$ lbs. $= 0.000$ tons.

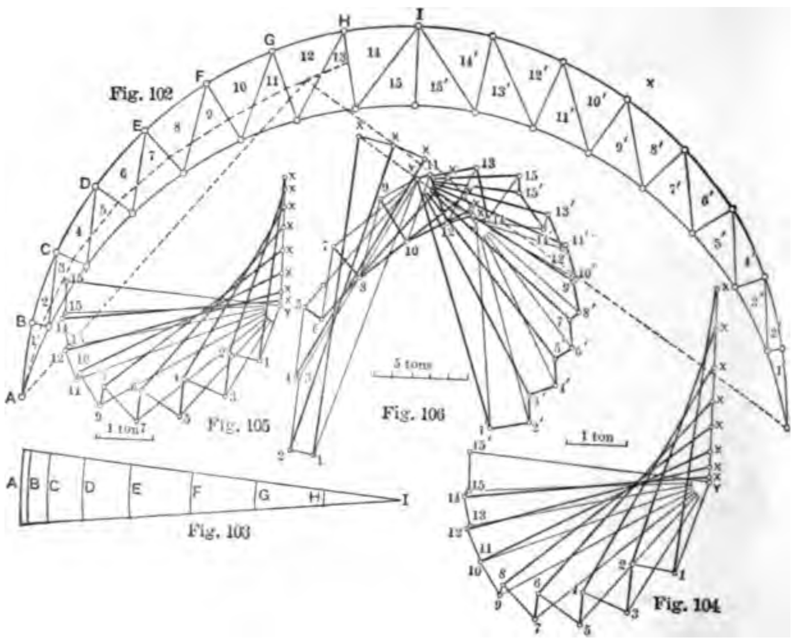
166. Total Load on Half Truss.—

Permanent $= 0.682 + 0.647 + 0.590 + 0.479 + 0.415 + 0.305 + 0.190 + \dfrac{0.097}{2} = 3.36$ T.

Snow $= 0.172 + 0.322 + 0.360 + 0.415 + 0.414 + 0.315 + 0.172 + \dfrac{0.041}{2} = 2.20$ T.

Wind, resultant found graphically.

167. Stress Diagrams.—The permanent stress diagram in Fig; 104 and the snow stress diagram in Fig. 105 are at the same scale.



FIGS. 102–106.—Hemispherical Crescent Dome.

the wind stress diagram in Fig. 106 is at a smaller scale, because it would occupy a much greater area if drawn at the scale of the others. These are somewhat similar in general form to the stress diagrams of Example 10, though the loads diminish more rapidly toward the vertex of the dome.

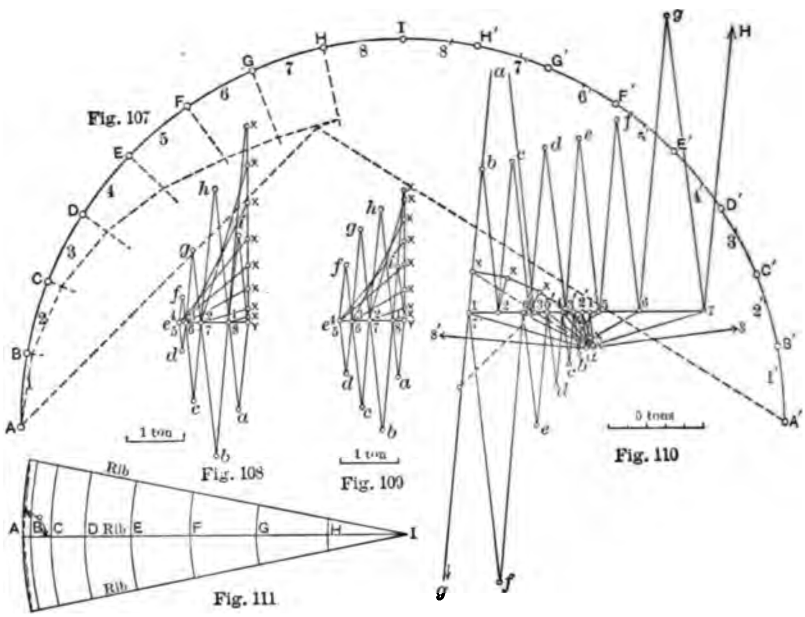
168. Stress Sheet for Example 11.—

Member.	*P*-stress.	*S*-stress.	*W*-stress *W*.	*W*-stress *L*.	Maximum.	Minimum.
X 1	− 6.2	− 3.2	−17.0	+11.3	−26.4	+ 5.1
X 2	− 5.6	− 3.0	−17.0	+11.3	−25.6	+ 5.7
X 4	− 5.3	− 3.5	−13.3	+10.0	−22.1	+ 4.7
X 6	− 5.2	− 3.8	− 9.9	+ 8.8	−18.9	+ 3.6
X 8	− 5.2	− 4.0	− 6.7	+ 7.7	−15.9	+ 2.5
X 10	− 5.0	− 4.0	− 3.7	+ 6.5	−12.7	+ 1.5
X 12	− 4.9	− 3.9	− 1.1	+ 5.4	− 9.9	+ 0.5
X 14	− 4.8	− 3.8	+ 1.1	+ 4.1	− 9.7	− 0.7
Y 1	+ 3.0	+ 1.1	+15.7	−13.8	+19.8	−10.8
Y 3	+ 3.5	+ 1.9	+12.4	−13.0	+17.8	− 9.5
Y 5	+ 4.1	+ 2.7	+ 9.0	−12.2	+15.8	− 8.1
Y 7	+ 4.5	+ 3.3	+ 5.5	−11.2	+13.3	− 6.7
Y 9	+ 4.7	+ 3.7	+ 2.1	−10.2	+10.5	− 5.5
Y 11	+ 4.8	+ 3.8	− 1.1	− 9.0	+ 8.6	− 4.2
Y 13	+ 4.8	+ 3.8	− 3.7	− 7.5	+ 8.6	− 3.7
Y 15	+ 4.7	+ 3.7	− 5.9	− 5.9	+ 8.4	− 1.2
1 2	+ 0.7	+ 0.6	+ 1.5	− 2.2	+ 2.8	− 1.5
3 4	+ 0.9	+ 0.8	+ 0.0	− 1.7	+ 1.7	− 0.8
5 6	+ 0.9	+ 0.9	− 1.2	− 1.2	+ 1.8	− 0.3
7 8	+ 0.8	+ 0.8	− 2.2	− 0.7	+ 1.6	− 1.4
8 10	+ 0.8	+ 0.7	− 2.7	− 0.3	+ 1.5	− 1.9
11 12	+ 0.7	+ 0.6	− 2.8	+ 0.2	+ 1.3	− 2.1
13 14	+ 0.7	+ 0.6	− 2.8	+ 0.7	+ 1.3	− 2.1
15 15	+ 0.8	+ 0.6	− 0.9	− 0.9	+ 1.4	− 0.1
2 3	− 0.3	+ 0.8	+ 3.7	− 1.3	− 2.4	+ 3.4
4 5	− 0.5	− 0.7	+ 3.9	− 1.4	− 2.6	+ 3.4
6 7	− 0.3	− 0.6	+ 4.1	− 1.5	− 2.4	+ 3.8
8 9	− 0.2	− 0.3	+ 4.2	− 1.6	− 2:1	+ 4.0
10 11	− 0.0	− 0.1	+ 3.9	− 1.7	− 1.8	+ 3.9
12 13	+ 0.1	+ 0.1	+ 3.4	− 2.1	− 2.3	+ 3.3
14 15	+ 0.1	+ 0.1	+ 2.9	− 2.4	− 2.6	+ 2.8

EXAMPLE 12.—HEMISPHERICAL RING DOME.

169. Description.—Form and construction of the external surface same as those in Example 11. Its surface is supported by the same rafters and equidistant purlins, but these rest on 16 complete meridian ribs instead of trusses. By crossed rods below each area of roof between purlins and ribs, deformation of the surface of the dome is prevented. A good example of this type of dome may be seen in the State Fair buildings at Springfield, Ill.

170. Programme.—Span, rise of rib, covering, sheathing, steel



FIGS. 107–111.—Hemispherical Ring Dome.

rafters, purlins, and ribs are as in Example 11. The ribs represent the upper chords of the trusses. Location and wind pressure are as in the last example. (Art. 163.)

171. Dimensions.—Inclinations at apexes, apex areas, are as before. (Art. 164.)

172. Apex Loads.—Permanent, snow, and wind apex loads are as in the last example. (Art. 165.)

173. Total Loads on Half Truss.—Also the same as for Example 11. (Art. 166.)

174. Stress Diagrams.—The semicircle $A\ I\ A'$, Fig. 107, is first drawn to represent the rib and is divided into 16 equal panels, the apexes A, B, C, etc., being at these points of division. Laying off in Fig. 108 the permanent loads as before, the y-point will be at the lower end of the load line for the half rib. Then drawing a parallel to the chord of $A\,B$ through upper end of load line, this cuts the horizontal through y at 1, obtaining the stress $y\,1$ acting horizontally and radially at the junction of the rib $A\,B$ with the horizontal base ring at A. It produces tension at A in the ring, which may be determined as follows.

175. Stresses in Ring Purlins.—Fig. 111 represents a middle and two adjacent half ribs in plan. Draw the chords of the purlin arcs at the base and measure angle $\beta°$ between purlin chord and middle rib on the plan. Then in Fig. 108 draw $y\,a$ and $1\,a$, making the angles $1\,y\,a$ and $1\,a\,y$ each $= \beta°$. These lines intersect at a and $y\,a$ or $1\,a$ = tensile stress in the chord of the purlin at A. The actual stress in the purlin ring must of course be greater because curved, but allowance can be made for this in dimensioning this ring.

Since the chords of all the purlins are parallel in each bay of the dome, the completion of the permanent and snow stress diagrams in Figs. 108 and 109 presents no difficulties.

When the x-stress lines cut the horizontal through y nearer the y-point than the one last drawn, the nature of the stress in the ring changes to compression and the triangles are then drawn above the horizontal through y to indicate this change.

For the wind stress diagram in Fig. 110, the wind loads are laid off on the load line as before, the dividing or y-point on their resultant being found by applying the equilibrium polygon, and a horizontal through the y-point is the location of the horizontal radial forces at the apexes. The chord purlin stress lines are drawn as before; some are found to be in compression and others in tension. It is interesting to observe that the values of these stress lines pass through infinity at the vertex of the dome, offering a good reason for an opening there, a skylight or a raised lantern.

176. Stress Sheet for Example 12.—

Member.	*P*-stress.	*S*-stress.	*W*-stress *W*.	*W*-stress *L*.	Maximum.	Minimum.
X 1	− 3.4	− 2.3	− 2.0	+ 2.0	− 7.7	− 1.4
X 2	− 2.9	− 2.2	− 1.7	+ 2.1	− 6.8	− 0.8
X 3	− 2.3	− 2.0	− 1.1	+ 2.3	− 5.4	− 0.6
X 4	− 1.9	− 1.8	− 0.1	+ 2.6	− 3.8	+ 0.7
X 5	− 1.6	− 1.5	− 1.3	+ 3.1	− 4.4	+ 1.5
X 6	+ 1.2	+ 1.1	− 3.2	+ 4.2	− 2.0	+ 5.5
X 7	+ 0.9	+ 0.7	− 6.4	+ 6.8	− 5.5	+ 8.4
X 8	+ 0.3	+ 0.2	−20.3	+20.3	−20.0	+20.8

177. Stresses in Purlin Chords.—

Member.	*P*-stress.	*S*-stress.	*W*-stress *W*.	*W*-stress *L*.	Maximum.	Minimum.
At *A*	+ 1.6	+ 1.1	+16.7	−14.4	+28.4	−12.8
B	+ 2.5	+ 2.0	+ 7.5	− 2.1	+12.0	+ 0.4
C	+ 1.4	+ 1.6	+ 8.1	− 2.3	+11.1	− 0.9
D	+ 0.5	+ 0.9	+ 8.7	− 3.1	+10.1	− 2.6
E	+ 0.0	+ 0.0	+ 9.5	− 4.0	+ 9.5	− 4.0
F	− 0.5	− 0.9	+11.2	− 6.5	+10.7	− 7.9
G	− 1.4	− 1.6	+17.4	−14.0	+16.0	−17.0
H	− 2.5	− 2.0	+72.8	−70.8	+70.3	−75.3

Example 13.—Cantilever Trusses

178. Different Types.—Trusses of this kind are attached to an external wall to support a projecting shed roof over a freight or delivery platform. The truss may be supported in various ways.

1. Ends of both upper and lower chords are directly bolted to the wall as in Figs. 112, 113, and 116.

2. End of lower chord fixed to wall and entire load supported there; end of upper chord only held by a horizontal connection only able to resist a horizontal force. Fig. 114.

3. End of upper chord fixed and entire load supported there by wall; end of lower chord merely abuts against the wall. Fig. 115.

4. Outer end of truss supported by a strut extending to wall; upper chord fixed to wall; end of lower chord free. Fig. 117.

5. Truss partially supported by strut from wall to an apex of lower chord; upper chord fixed to wall; lower chord free. Fig. 118.

6. Outer end of truss supported by rod extending to wall; lower chord fixed to wall; upper chord free. Fig. 119.

7. Truss partially supported by rod from wall to an apex of upper chord; lower chord fixed to wall; upper chord free. Fig. 122.

When the truss is partially supported by a rod or strut extending to the wall, the ends of both chords cannot be fixed to it, for the problem then becomes indeterminate. It is then best to fix the end of the chord most distant from the rod or strut, thus insuring greater stability of truss.

179. Programme.—Type of truss similar in all cases treated here, excepting that reversed diagonals are used in some of the cases.

Span, 30 ft.; rise at wall, 10 ft.; 5 panels; trusses, 12 ft. on centres; materials, steel purlins and trusses; no rafters; covering of painted tin on 7/8″ shortleaf pine sheathing; location at Cairo. Ill., latitude about 37° north; ordinary exposure.

180. Dimensions.—

$$\text{Tan } i^\circ = \frac{10}{30} = 0.3333 = \tan 18.4^\circ.$$

$$l = \frac{30}{5} = 6.00;\ l' = \frac{6.00}{\cos 118.4^\circ} = 6.32 \text{ ft.}$$

$A = 6.32 \times 12.00 = 75.84$ sq. ft. = apex area.

181. Apex Loads.—

Truss (for twice span) $= \frac{60}{25} + \frac{3600}{12600} =$ 2.686 lbs. per horizontal sq. ft.

Snow = 2.5 (37° − 35°) = 5.00 lbs. per horizontal sq. ft.

Wind $= \frac{2}{3} \times 18.4^\circ =$ 12.30 lbs. per inclined sq. ft.

$P = 75.84\,(2 + 3 + 0 + 4 + 2.686 \cos 18.4^\circ) = 875 \times = 0.438$ ton.

$S = 75.84\ (5.00 \cos 18.4^\circ) = 360 = 0.180$ ton.

$W = 75.84 \times 12.30 = 933 = 0.467$ ton.

182.—Total Loads on Truss.—Permanent = 0.438 × 5 = 2.19 tons.

Snow = 0.180 × 5 = 0.90 ton.

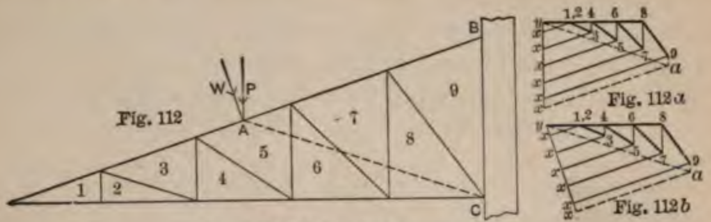
Wind = 0.467 × 5 = 2.34 tons.

183. Stress Diagrams.—Case 1. Truss fixed to wall at B and C. Figs. 112, 112 *a*, 112 *b*.

The resultants of all P and of all W apex loads act at A. Join $A\,C$. In Figs. 112 *a* and 112 *b*, $y\,a$ is parallel to $A\,C$ and $x\,a$ to $A\,B$. Also $y\,a$ = force acting at C and $x\,a$ that at B. The S

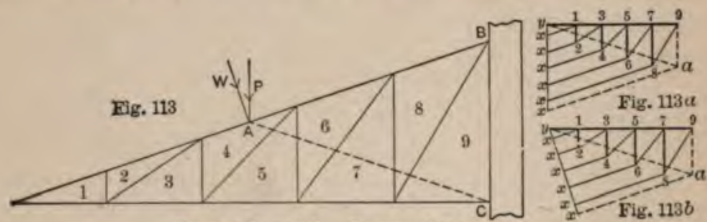
stresses can be found from the P stresses by the proportion: 2.19 : 0.90 : : P stress : S stress.

184. Case 1a.—Truss and loads as before, but the diagonals



Figs. 112–112 *b*.–Cantilever Truss.

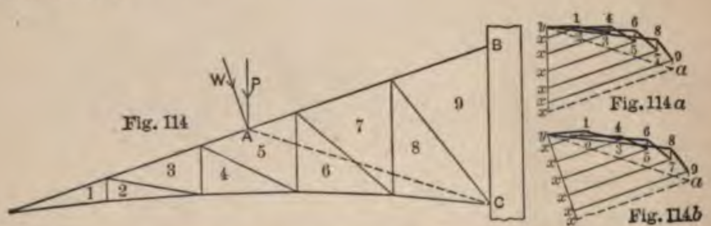
are reversed. Figs. 113, 113 *a*, 113 *b*. Drawn in the same manner, producing different forms of stress diagrams.



Figs. 113–113 *b*.—Reversed Diagonals.

185. Case 1b.—Truss fixed at B and C; lower chord curved. Figs. 114, 114 *a*, and 114 *b*.

Drawn in the same manner as Case 1, excepting that stress lines for lower chord radiate from y in the stress diagrams and are parallel to its members.

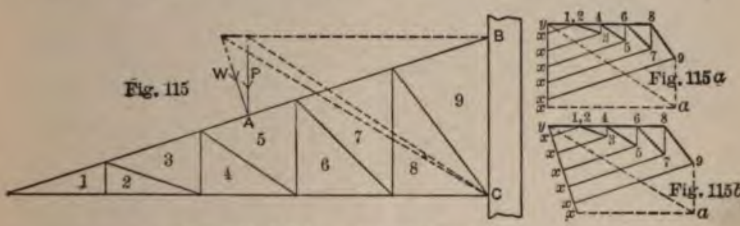


Figs. 114–114 *b*.—Chord Curved.

186. Case 2.—Truss fixed and supported at C, only held horizontally at B. Figs. 115, 115 *a*, and 115 *b*.

Resultant W is produced to intersect a horizontal through B at D, and $D\,C$ are joined, being line of action of component of W acting at C. The component of P is found in the same manner. In Figs. 114 a and 114 b, $x\,a$ is drawn horizontally and = force acting at B; $y\,a$ is parallel to $E\,C$ and $D\,C$ and = force acting at C.

187. Case 3.—Truss fixed and supported at B, end of lower chord abutting against wall at C. Figs. 116, 116 a, 116 b.

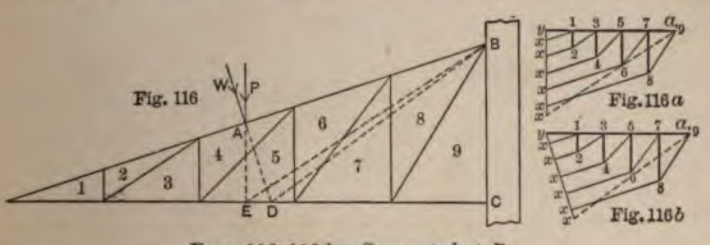


Figs. 115–115 b.—Supported at C.

Resultant W is produced to intersect horizontal lower chord at D; join $D\,B$, which is line of action of W component at C. $E\,C$ is line of action of component of P and is found in same manner. In the stress diagrams, $y\,a$ = horizontal force acting at C, and $x\,a$ = inclined force applied at B.

188. Case 4.—Truss fixed to wall at B; lower chord loose at C; end F of truss supported by strut attached to wall at G. Figs. 117, 117a, 117b.

Resultant W is here produced to intersect strut at D, which is joined with B. $E\,B$ is then found in the same manner. In P stress diagram, Fig. 117 a, draw $y\,a$ parallel to strut $F\,G$ and $x\,a$ parallel to



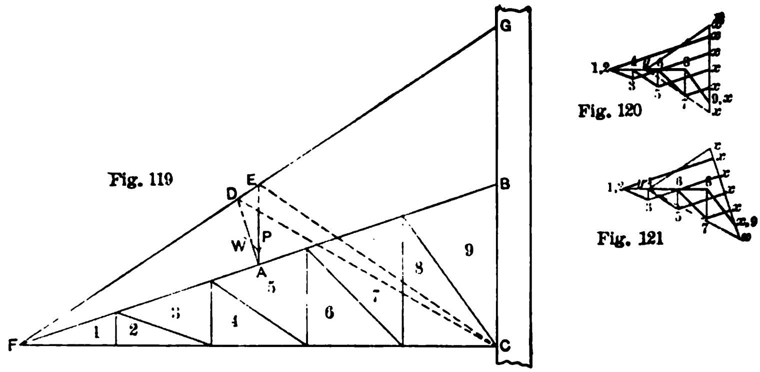
Figs. 116–116 b.—Supported at B.

$E\,B$; through a draw horizontal a 1, and y falls at its intersection with the load line $x\,x$. The x lines on left of load line are −, those on the right being +; the reverse is true of the y lines. The W stress diagram is drawn in the same manner in Fig. 117 b.

[illegible] parallel [illegible] *F G* [illegible] which is then drawn [illegible] while they are — [illegible] drawn in like manner.

191. Case 7.—Truss fixed to wall at *C*; *B* is free; truss partly supported by rod *F G*. Figs. 122, 123, 124.

Resultants *W* and *P* are produced to cut *F G* extended at *D* and *E*, which are then joined with *C*. In the *P* stress diagram, draw



Figs. 119–121.—Supported by Rod.

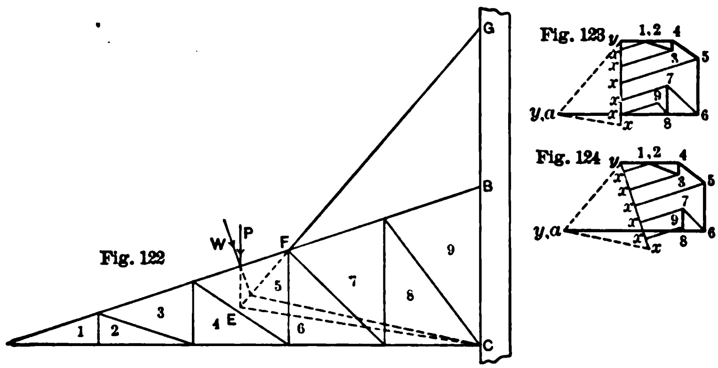
y a parallel to *F G*, *x a* parallel to *E C*; also draw horizontal *y*-lines through *y* and *a*; *y* 6 and *y* 8 are measured on the latter. The *W* stress diagram is similar.

192. Comparative Table of Maximum Stresses in Members.—

Case.	1	1 *a*	1 *b*	2	3	4	5	6	7
X 2 or 1	+ 1 7	+ 1.7	+ 2.3	+ 1.8	+ 1.8	− 2.9	+ 1.8	− 6.5	+ 1.7
X 2 or 3	+ 3.1	+ 1.5	+ 4.2	+ 3.4	+ 1.6	− 3.0	+ 1.6	− 4.9	+ 3.3
X 4 or 5	+ 5.2	+ 3.2	+ 5.8	+ 5.0	+ 3.2	− 1.4	+ 3.2	− 3.2	+ 5 0
X 6 or 7	+ 6.6	+ 4.8	+ 7.3	+ 6.6	+ 4.8	− 0.3	+ 3.8	− 0.0	+ 2.4
Y 1	− 1.6	− 1.7	− 2.4	− 1.8	− 1.8	+ 2.7	− 1.8	+ 2.2	− 1.8
Y 3 or 2	− 1.6	− 3.5	− 2.4	− 1.8	− 3 3	+ 1.0	− 3.4	+ 2.2	− 1.8
Y 5 or 4	− 3.3	− 5.0	− 4.3	− 3.5	− 5.0	− 0 9	− 5.1	+ 0.5	− 3.3
Y 7 or 6	− 5.1	− 6.8	− 5.8	− 5.1	− 6.8	− 2.3	− 0.0	− 1 1	− 9.0
Y 9 or 8	− 6.7	− 8.5	− 7.4	− 6.9	− 8.4	− 4.1	− 0.0	− 2.8	− 7.2
1 2	+ 0.0	− 1.1	− 0.2	+ 0.0	− 1.2	− 1.1	− 1 1	+ 0.0	+ 0.0
3 4	+ 0.6	− 1.7	+ 0.2	+ 0.6	− 1.8	− 1.5	− 1.8	+ 0.5	+ 0.6
5 6	+ 1.2	− 2.3	+ 0.5	+ 1.2	− 2.3	− 2.3	− 2.3	+ 1.2	− 3.4
7 8	+ 1.7	− 2.8	+ 0.7	+ 1.8	− 2.8	− 2.8	− 0.0	+ 1.8	− 1.7
2 3	− 1.8	+ 2.0	− 2.0	+ 1.8	+ 1.9	+ 2.0	+ 2 0	− 1.7	− 1.7
4 5	− 2.0	+ 2.3	− 1.9	− 1.9	+ 2.4	+ 2.3	+ 2.5	− 2.0	− 2.0
6 7	− 2.5	+ 2.8	− 1.9	− 2.2	+ 2.8	+ 2.8	+ 1.5	− 2.5	+ 2.5
8 9	− 2.8	+ 3.2	− 1.8	− 2.7	+ 3.2	+ 3.2	+ 0.0	− 2.9	+ 0.7
Rod or strut	0.0	0.0	0.0	0.0	0.0	− 4.4	− 5.5	+ 4.6	+ 5.5

EXAMPLE 14.—TRUSS WITH CANTILEVER AT EACH END.

195. Description.—This truss is supported only at the intermediate apexes C and D of the lower chord on stable walls or strongly anchored piers or columns, as in Fig. 125. The stability of such walls, piers, or columns is to be studied later, but it is here assumed



FIGS. 122–124.—Supported by Rod.

to be sufficient. Projecting beyond the walls at each end of truss are the cantilevers or overhangs $A\ C$ and $D\ B$, as extensions of the main roof truss $C\ D$. Such a roof would be suitable for a railway train shed, a freight or express building, a warehouse for temporary storage, etc. A comparison of the stress sheet and stress diagrams with those for a truss of equal span and supported at its ends A and B proves this to be a very economical type of roof truss.

196. Programme.—Type as in Fig. 125: total span, 200 ft., composed of two cantilevers of 50 ft. each and a middle span of 100 ft., supported on solid walls or anchored columns; rise, 40 ft. at middle; 20 ft. at each support; 16 panels, of equal length; materials, steel; trusses, 20 ft. on centres; covering of painted tin on 7/8-inch shortleaf pine sheathing; steel rafters, purlins, and trusses; location at Omaha, latitude about 41 1/5° north; medium exposure.

197. Dimensions.—Tan $i = \dfrac{40}{100} = 0.4000 = \tan\ 21.8°$.

$$l = \frac{200}{16} = 12.5 \text{ ft.};\ l' = \frac{12.5}{\cos 21.8°} = 13.46 \text{ ft.}$$

$$A = 13.46 \times 20.00 = 269.2 \text{ sq. ft.} = \text{apex area.}$$

198. Apex Loads.—

$$\text{Truss} = \frac{200}{25} + \frac{200^2}{12600} = 11.175 \text{ lbs. per horizontal sq. ft.}$$

$$\text{Snow} = 2.5\ (41.2° - 35°) = 15.6 \text{ lbs. per horizontal sq. ft.}$$

$$\text{Wind} = \frac{8}{9} \times 21.8° = 19.38 \text{ lbs. per inclined sq. ft.}$$

$P = 269.2\ (2 + 3 + 4 + 3 + 11.175 \cos 21.8°) = 6025$ lbs. $=$ 3.013 tons.

$S = 269.2\ (15.6 \cos 21.8°) = 3898$ lbs. $= 1.949$ tons.

$W = 269.2 \times 19.38 = 5217$ lbs. $= 2.609$ tons.

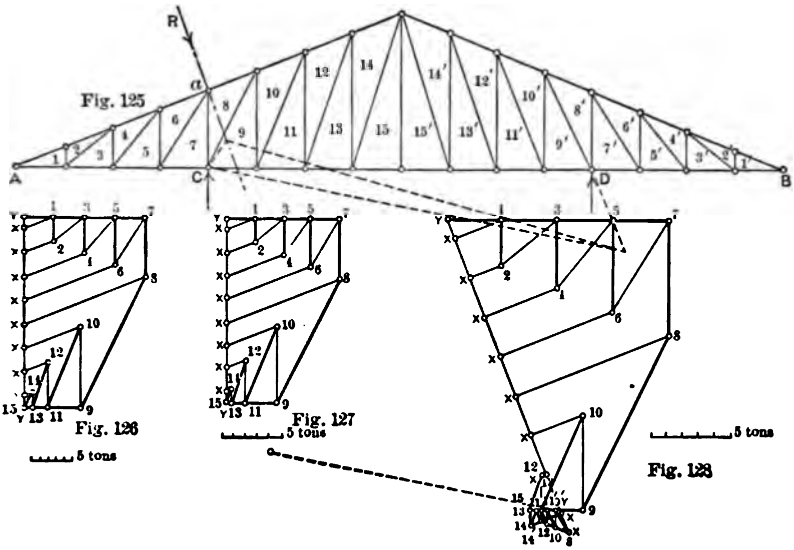
199. Total Loads on Half Truss.—

Permanent $= 3.013 \times 8 = 24.11$ tons.

Snow $= 1.949 \times 8 = 15.59$ tons.

Wind $= 2.609 \times 8 = 20.87$ tons.

200. Stress Diagrams.—The permanent stress diagram in Fig. 126 is drawn in the same manner as for the cantilever truss in Example



Figs. 125–128.—Truss with Overhangs at Ends.

13, until the point 8 is found. The y-point is then transferred to the lower end of the load line for the half truss, and the diagram is then completed as indicated. Since there is no stress found in the member 15 16, the point 16 should fall on the y, if the work is correct.

The snow stress diagram in Fig. 127 is entirely similar in form.

For the wind stress diagram in Fig. 128, the resultant R of all wind loads on the left side of the roof is first located, which is here at a, vertically above the support C. This resultant is then resolved into its parallel components passing through the two supports C and D, thus obtaining the dividing point on the load line for wind. Then $y\,y$ represents the reaction at C and $x\,y$ is that at D. The wind stress diagram is readily completed, though the stresses in the members are quite small for the leeward half of the truss, and no wind stresses occur in the members beyond 8′ 9′.

201. Stress Sheet for Example 14.—

Member.	*P*-stress.	*S*-stress.	*W*-stress *W*.	*W*-stress *L*.	Maximum.	Minimum.
X 1	+ 4.3	+ 2.7	+ 3.3	+ 0.0	+10.3	+ 4.3
X 2	+ 4.3	+ 2.7	+ 2.2	+ 0.0	+99.2	+ 4.3
X 4	+ 8.3	+ 5.2	+ 4.9	+ 0.0	+18.4	+ 8.3
X 6	+12.4	+ 7.6	+ 6.7	+ 0.0	+26.9	+12.4
X 8	+16.5	+10.4	+ 9.6	+ 0.0	+36.5	+16.5
X 10	+ 7.5	+ 4.6	+ 3.0	− 0.9	+15.1	+ 6.6
X 12	+ 3.0	+ 1.7	− 0.5	− 1.5	+ 6.2	+ 1.5
X 14	+ 0.8	+ 0.3	− 2.2	− 2.0	+ 1.1	− 1.1
Y 1	− 4.0	− 2.4	− 3.5	− 0.0	− 9.9	− 4.0
Y 3	− 7.7	− 4.8	− 7.0	− 0.0	−19.5	− 7.7
Y 5	−11.5	− 7.3	− 9.6	− 0.0	−28.4	−11.5
Y 7	−15.2	− 9.6	−13.4	− 0.0	−38.2	−15.2
Y 9	− 7.0	− 4.3	− 1.1	+ 0.2	−12.4	− 6.8
Y 11	− 2.7	− 1.5	+ 1.2	+ 0.8	− 4.2	− 1.5
Y 13	− 0.7	− 0.3	+ 1.9	+ 1.2	− 1.0	+ 1.2
Y 15	− 0.1	− 0.0	+ 1.5	+ 1.5	+ 1.5	− 0.1
1 2	− 3.0	− 2.0	− 2.9	− 0.0	− 7.9	− 3.0
3 4	− 4.6	− 3.0	− 4.2	− 0.0	−11.8	− 4.6
5 6	− 6.0	− 3.9	− 6.0	− 0.0	−15.9	− 6.0
7 8	− 7.6	− 4.9	− 7.3	− 0.0	−19.8	− 7.6
9 10	+10.2	+ 6.5	+ 5.6	+ 1.3	+22.3	+10.2
11 12	+ 5.6	+ 3.5	+ 1.8	+ 1.1	+10.9	+ 5.6
13 14	+ 1.8	+ 1.1	− 1.3	+ 0.9	+ 3.8	+ 1.8
15 15	+ 0.0	+ 0.0	+ 0.0	+ 0.0	+ 0.0	+ 0.0
2 3	+ 4.8	+ 3.1	+ 4.5	+ 0.0	+12.4	+ 4.8
4 5	+ 6.0	+ 3.8	+ 5.0	+ 0.0	+14.8	+ 6.0
6 7	+ 7.1	+ 4.6	+ 7.1	+ 0.0	+18.8	+ 7.1
8 9	−18.6	−12.0	−11.7	− 1.9	−42.3	−18.6
10 11	−11.1	− 7.1	− 6.0	− 1.5	−25.2	−11.1
12 13	− 5.9	− 3.8	− 1.9	− 1.2	−11.6	− 5.9
14 15	− 1.8	− 1.1	− 1.3	− 1.0	− 4.2	− 1.8

Example 15.—Three-hinged Truss

202. Description.—This truss is hinged at each support and at the ridge by joint pins, Fig. 129, and it requires no other provision for expansion or contraction by changes of temperature, since the ridge rises or falls slightly during heat or cold. This type of truss is quite economical for large spans, but the horizontal components of the inclined reactions at the supports require these supports to be connected by tie-rods beneath the floor or to be fixed to pile or massive masonry foundations. Inclined stone or concrete piers with stepped footings would also serve, provided that the reactions at the supports do not vary much from the axes of such piers.

203. Programme.—Truss of type as in Fig. 129; span, 100 ft.; rise of side verticals, 30 ft.; of ridge, 20 ft., making a total height of 50 ft.; 3 panels in side verticals and 10 panels in inclined roof; material, steel; trusses, 20 ft. on centres; covering of painted tin on 7/8 white pine sheathing on both walls and roof; steel rafters, purlins, and trusses; location at Toronto; latitude about 43.7° north; medium exposure.

204. Dimensions.—$\text{Tan } i = \frac{20}{50} = 0.4000 = \tan 21.8°.$

$l = 10.00$ ft.; $l' = \frac{10.00}{\cos 21.8°} = 10.77$ ft.

$A = 10.77 \times 20.00 = 215.4$ sq. ft. for apex area of roof.

$A = 10.00 \times 20.00 = 200.0$ sq. ft. for apex area of wall.

205. Apex Loads.—

$\text{Truss} = \frac{10}{25} + \frac{100^2}{12600} = 4.794$ lbs. per horizontal sq. ft.

$\text{Snow} = 2.5\ (43.7° - 35°) = 21.67$ lbs. per horizontal sq. ft.

$\text{Wind} = \frac{8}{9} \times 21.8° = 19.38$ lbs. per inclined sq. ft.

P (wall) $= 200\ (2 + 3 + 4 + 3 + 4.794) = 3359$ lbs. $= 1.680$ tons.

P (roof) $= 215.4\ (2 + 3 + 4 + 3 + 4.794 \cos 21.8°) = 3543$ lbs. $= 1.772$ tons.

$S = 215.4\ (21.67 \cos 21.8°) = 4330$ lbs. $= 2.165$ tons.

W (wall) $= 200 \times 40.00 = 8000$ lbs. $= 4.000$ tons.

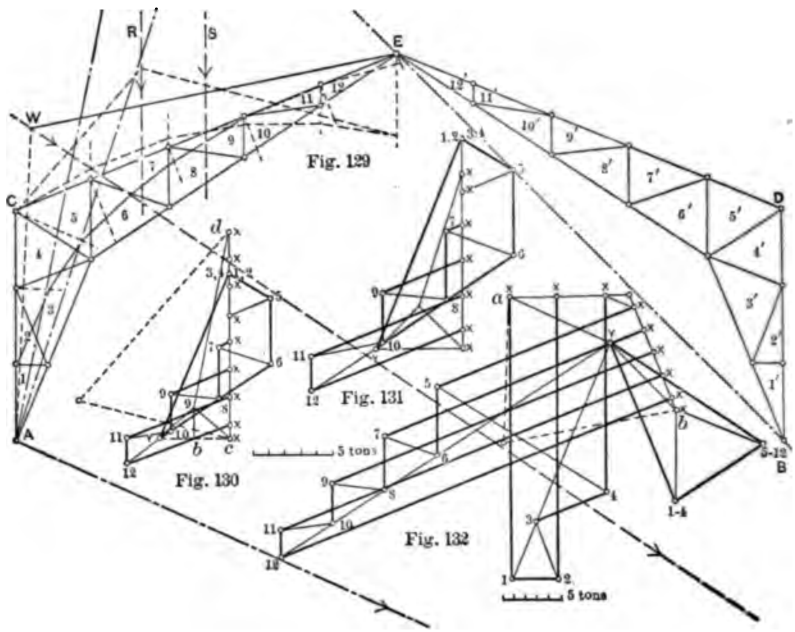
W (roof) $= 215.4 \times 19.38$ lbs. $= 4174$ lbs. $= 2.087$ tons.

206. Total Loads on Half Truss.—

Permanent = 1.680 × 2 1/2 + 1.772 × 5 = 13.06 tons.
Snow = 2.165 × 5 = 10.83 tons.
Wind (wall) = 4.000 × 2 1/2 = 10.00 tons.
Wind (roof) = 2.087 × 5 = 10.44 tons.

The construction of the walls is here assumed to be similar to that of the roof.

207. Stress Diagrams for Example 15.—For a truss with three hinges and supporting permanent or snow loads, it is first necessary



FIGS. 129–132.—Truss with Three Hinges.

to consider the half truss *A C E* in Fig. 129. The resultant *R* of all the permanent loads supported by this half truss is located by the equilibrium polygon in Fig. 129, and it is produced upward to intersect the line *E B* extended, which connects the hinges *E* and *B*. This intersection is then joined with the hinge *A*. At the intersection of *R* and the line *E B*, the resultant is resolved into two components, one acting along the line just drawn to *A*, the other passing along the line *E B* to the hinge *B*, since the component

transmitted to B must pass along the straight line $E\,B$, connecting the hinges E and B.

The permanent load line is next laid off vertically in Fig. 130; a parallel to the component acting at A is drawn through its upper end, and a parallel to the component transmitted to B through its lower end, these lines intersecting at a, Fig. 130; a horizontal is then drawn through the lower end of load line, next the vertical $a\,b$ is drawn, and $b\,y$ is made equal to $b\,c$. Join $a\,y$, which then = the component passing from the permanent load on the right half of the truss through E to A, and $a\,d$ = the component from the half truss $A\,C\,E$, acting at A. Joining $d\,y$, this represents the entire reaction at A for the permanent loads on the entire truss. The permanent stress diagram is then easily completed by proceeding from A to C, then beginning again at E, joining the two ends of the diagram near C, where the last stress line must be parallel to the corresponding member, if the work is correct.

Since there is no snow on the side walls, the resultant S of all snow loads on the truss $A\,C\,E$ must act vertically at the centre of the middle panel of the roof truss. Producing this resultant S upward to intersect $E\,B$ extended as before, this intersection is joined with A. The snow stress diagram is then completed in the same manner as the permanent stress diagram.

Since the walls are here assumed to be enclosed, the maximum wind pressure of 40 lbs. there acts on this vertical surface $A\,C$. The resultant W of all wind loads on wall and roof of the half truss is then located and produced to intersect $E\,B$, this intersection here falling below the limits of the drawing. The intersection is then joined with A, obtaining the line of the reaction at A, since there being no wind loads on the other half truss, no component can be transmitted to A through E.

In Fig. 132, the wind loads are laid off from a to b'. Through a is then drawn $a\,y$ parallel to reaction at A, and through b, $b\,y$ parallel to $E\,B$, the location of the reaction at B. Their intersection then becomes the y-point of the wind stress diagram. and the diagram is readily completed.

208. Stress Sheet for Example 15.—

Member.	*P*-stress.	*S*-stress.	*W*-stress *W*.	*W*-stress *L*.	Maximum.	Minimum.
X 1	− 2.5	+ 1.8	−23.0	+ 7.8	−25.5	+ 7.1
X 2	− 0.8	+ 1.8	−23.0	+ 7.8	−23.8	+ 8.8
X 4	+ 0.9	+ 1.8	−16.6	+ 7.8	−15.7	+10.5
X 5	+ 2.6	+ 2.2	−17.2	+ 7.6	−14.6	+12.4
X 7	− 1.0	− 1.6	−22.3	+ 7.6	−24.9	+ 6.6
X 9	− 4.6	− 5.8	−27.7	+ 7.6	−37.8	+ 3.6
X 11	− 8.0	−10.0	−33.0	+ 7.6	−51.0	− 0.4
X 12	− 8.0	−10.0	−33.8	+ 7.6	−51.8	− 0.4
Y 1	−11.4	−13.6	+20.8	−14.7	−39.7	+ 9.4
Y 3	−11.4	−13.6	+15.6	−14.7	−39.7	+ 4.2
Y 6	− 8.1	− 9.6	+16.5	−15.4	−33.1	+ 8.4
Y 8	− 4.2	− 4.7	+21.5	−15.4	−24.3	+11.4
Y 10	− 0.3	− 0.0	+26.4	−15.4	−15.7	+26.1
Y 12	+ 4.0	+ 4.7	+31.2	−15.4	−11.4	+39.9
1 2	− 0.0	+ 0.0	− 4.0	− 0.0	− 4.0	+ 0.0
3 4	− 0.0	+ 0.0	− 6.2	− 0.0	− 6.2	+ 0.0
5 6	− 4.4	− 5.5	− 5.6	− 0.0	−15.5	− 4.4
7 8	− 3.5	− 4.4	− 4.5	− 0.0	−12.4	− 3.5
9 10	− 2.6	− 3.2	− 3.4	− 0.0	− 9.2	− 2.6
11 12	− 1.8	− 2.2	− 2.2	− 0.0	− 6.2	− 1.8
2 3	− 0.0	− 0.0	+ 5.3	− 0.0	+ 5.3	+ 0.0
4 5	− 2.9	− 3.1	+16.4	− 8.6	−14.6	+13.5
6 7	+ 3.6	+ 4.4	+ 4.6	− 0.0	+12.6	+ 3.6
8 9	+ 3.3	+ 3.9	+ 4.1	− 0.0	+11.3	+ 3.3
10 11	+ 3.3	+ 3.9	+ 4.1	− 0.0	+11.3	+ 3.3
Tie-rod	+ 4.4	+ 5.4	+ 5.8	− 0.0	+15.6	+ 4.4

EXAMPLE 16.—THREE-HINGED ARCH WITH CANTILEVERS AT ENDS

209. Description.—This truss is superior to that of Example 15 in appearance and the cantilevers would here balance the half arches on the supports *M* and *N*, so that no portion of the permanent or snow loads on the left half truss would be transmitted along the line *L N* to the support *N*. It is here assumed that the vertical wall *M B* is open and that the vertical surface *H G* is enclosed. Such a truss would be very suitable for a large train shed covered by a single roof. That of the I. C. R., at 12th St., Chicago, is of this type, but with shorter cantilevers at sides, and the trusses are spaced in pairs with no apparent advantage resulting therefrom.

210. Programme.—Truss of type as in Fig. 133; central arch, 100 ft. span and radius; cantilevers of 50 ft. span each; 25 ft. clear height from floor to cantilevers, which have 20 ft. rise; rise of middle roof, 20 ft.; 16 equal horizontal panels; material, steel; covering of painted tin on 7/8-inch longleaf pine sheathing; trusses, 25 ft. on centres; location at Buffalo; latitude about 42 2/3° north; ordinary exposure; sides open below cantilevers only.

211. Dimensions.—Tan $i = \frac{20}{50} = 0.4000 = \tan 21.8°$, for all roofs.

$$l = \frac{200}{16} = 12.5 \text{ ft.}; \; l' = \frac{12.5}{\cos 21.8°} = 13.46 \text{ ft.}$$

$A = 13.46 \times 25.00 = 336.5$ sq. ft. = apex and purlin areas.

212. Apex Loads.—Assume the truss to have the same weight as one of 100 ft. span, as this would probably be sufficient.

$$\text{Truss} = \frac{100}{25} + \frac{100^2}{12600} = 4.794 \text{ lbs. per horizontal sq. ft.}$$

Snow $= 2.5\ (42\ 2/3° - 35°) = 19.17$ lbs. per horizontal sq. ft.

$$\text{Wind} = \frac{2}{3} \times 21.8° = 14.53 \text{ lbs. per inclined sq. ft.}$$

Wind = 30 lbs. per sq. ft. of vertical wall above cantilevers.

It is here best to average the weight of the truss for joints of upper chord only.

Total weight of truss including cantilevers $= 4.794 \times 25 \times 200 =$ 23970 lbs.

Counting 25 apexes in the upper chords of entire truss, $\frac{23970}{25} =$ 959 lbs. per apex of upper chords.

P at A and B for truss only = 959 lbs. = 0.480 tons.

P at C and $L = \frac{336.5}{2}(2 + 4 + 4 + 3) + 959 = 3146$ lbs. = 1.573 tons.

P at D, E, F, I, J, and $K = 336.5\ (2 + 4 + 4 + 3) + 959 = 5333$ lbs. = 2.667 tons.

P at G and H $= \left(\frac{336.5}{2} + \frac{535.0}{2}\right)(2 + 4 + 4 + 3) + 959 = 6623$ lbs. $= 3.312$ tons.

S at C, G, H, and $L = \frac{336.5}{2} \times 19.17 \cos 21.8° = 2995$ lbs. $=$ 1.498 tons.

S at D, E, F, I, J, and $K = 336.5 \times 19.17 \cos 21.8° = 5990$ lbs. $= 2.995$ tons.

W at C and $L = \frac{336.5}{2} \times 14.53 = 2446$ lbs. $= 1.223$ tons.

W at D, E, F, G, I, J, and $K = 336.5 \times 14.53 = 4890$ lbs. $= 2.445$ tons.

W at G and $H = \frac{21.4 \times 25.00}{2} \times 30.00 = 8026$ lbs. $= 4.013$ tons.

213. Total Loads on Half Truss.—

Permanent $= 2 \times 0.480 + 2 \times 1.573 + 6 \times 2.667 + 2 \times 3.312 = 26.73$ tons.

Snow $= 4 \times 1.498 + 6 \times 2.995 = 23.96$ tons.

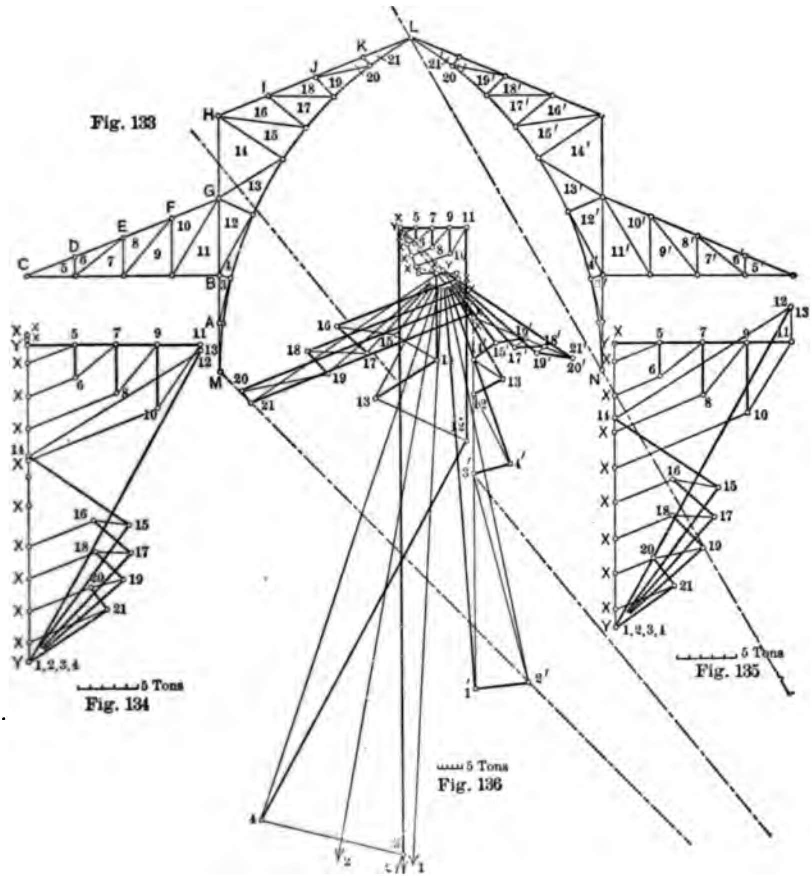
Resultant of wind loads must be obtained graphically.

214. Stress Diagrams.—Since the permanent loads are equal on both sides of the vertical drawn through the support M and balance each other about it, that vertical will contain their resultant, which acts entirely at M. The P stress diagram is shown in Fig. 134. The y-point for the cantilever is found on the load line after passing the P loads at A and B as indicated. Beginning there, the diagram for cantilever is completed as in Example 13, Fig. 112. Then commencing again at L and using the lower end of load line as the y-point, complete the diagram for the upper portion of the truss, joining the parts of the stress diagram near the junction of the cantilever and arched portion. For this truss, the stress diagram is to be commenced at three points, M, C, and L, instead of but two, as in the previous examples. The stresses in $X\,1$ and $X\,2$ are here represented by the entire length of the load line in Fig. 134.

The snow loads at the apexes balance similarly about the vertical drawn through M, and this vertical coincides with their resultant. Hence the snow stress diagram in Fig. 135 is drawn in the same manner as Fig. 134, though appearing in a slightly different form, there being no snow loads on the vertical wall at $G\,H$, and the

permanent weight of truss being omitted, so that no loads appear at M and A.

The resultant of the wind loads here passes through the middle of the vertical $G H$ and is parallel to their resultant in Fig. 136, the wind stress diagram. Its intersection with the chord $L N$ here falls



FIGS. 133–136.—Truss with Three Hinges and Overhangs.

below the drawing, but is connected with M, locating the reaction at M. Draw a parallel to this reaction through the upper end of the load line and a parallel to $L N$ through the lower end; their intersection is then the y-point for the wind stress diagram, which is readily completed as before.

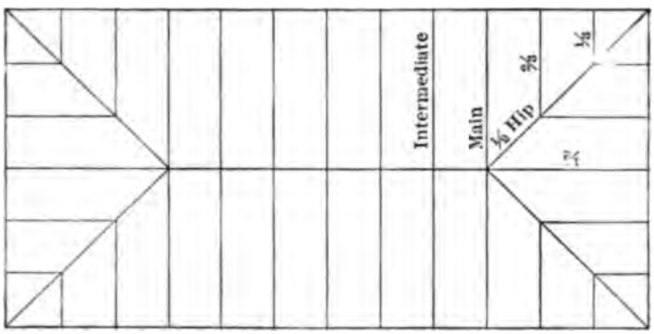
215. Stress Sheet for Example 16.—

Member.	*P*-stress.	*S*-stress.	*W*-stress *W*.	*W*-stress *L*.	Maximum.	Minimum.
X 1	−26.7	−24.0	−352.0	+76.5	−402.7	+49.8
X 3	−26.7	−24.0	−148.0	+34.2	−198.7	+ 7.5
X 5	+ 4.3	+ 4.0	+ 2.9	+ 0.0	+ 11.2	+ 4.3
X 6	+ 4.3	+ 4.0	+ 2.8	+ 0.0	+ 11.1	+ 4.3
X 8	+ 7.9	+ 8.0	+ 4.4	+ 0.0	+ 20.3	+ 7.9
X 10	+11.4	+12.1	+ 7.0	+ 0.0	+ 30.5	+11.4
X 14	+ 3.8	+ 6.0	− 20.5	+ 9.1	+ 18.9	−16.7
X 16	+ 5.6	+ 5.5	− 29.0	+12.5	+ 23.6	−23.4
X 18	+ 5.7	+ 5.7	− 38.0	+17.3	+ 28.7	−32.3
X 20	+ 4.7	+ 4.2	− 53.4	+25.5	+ 34.4	−48.7
X 21	+ 6.3	+ 6.1	− 52.7	+25.5	+ 37.9	−48.4
Y 1	− 0.0	− 0.0	+343.4	−87.0	+343.4	−87.0
Y 2	− 0.0	− 0.0	+343.0	−37.2	+343.0	−37.2
Y 4	− 0.0	− 0.0	+138.8	−44.7	+138.8	−44.7
Y 5	− 4.0	− 3.7	− 3.1	− 0.0	− 10.8	− 4.0
Y 7	− 7.3	− 7.4	− 6.4	− 0.0	− 21.1	− 7.3
Y 9	−10.6	−11.2	− 9.8	− 0.0	− 31.6	−10.6
Y 11	−13.9	−15.0	− 13.1	− 0.0	− 42.0	−13.9
Y 13	−28.8	−30.9	+ 36.2	−27.1	− 86.8	+ 7.4
Y 15	−13.9	−14.9	+ 19.0	−19.6	− 48.4	+ 5.1
Y 17	−12.2	−12.9	+ 25.8	−22.7	− 47.8	+13.6
Y 19	−10.1	−10.5	+ 35.9	−27.5	− 48.1	+25.8
Y 21	− 7.1	− 6.8	+ 52.2	−35.6	− 49.5	+45.1
1 2	− 0.0	− 0.0	+ 51.0	− 9.8	+ 51.0	− 9.8
3 4	− 0.0	− 0.0	+ 37.4	− 8.7	+ 37.4	− 8.7
5 6	− 2.7	− 3.0	− 2.7	− 0.0	− 7.4	− 2.7
7 8	− 4.0	− 4.5	− 4.0	− 0.0	− 12.5	− 4.0
9 10	− 5.3	− 6.0	− 5.5	− 0.0	− 16.8	− 5.3
11 12	− 0.5	− 3.2	− 49.2	+16.7	− 53.9	+16.2
12 13	− 0.0	− 0.0	+ 23.2	− 7.8	+ 23.2	− 7.8
14 15	−10.0	−10.7	+ 11.8	− 6.5	− 27.2	+ 1.8
16 17	− 4.1	− 4.7	− 8.9	+ 3.0	− 17.7	− 1.1
18 19	− 2.3	− 3.8	− 6.3	+ 1.8	− 12.4	− 0.5
20 21	− 2.6	− 2.9	− 2.2	+ 0.0	− 7.7	− 2.6
2 3	+ 0.0	+ 0.0	− 2.0	+42.4	+ 42.4	− 2.0
6 7	+ 4.3	+ 4.8	+ 4.2	+ 0.0	+ 13.3	+ 4.3
8 9	+ 5.2	+ 5.9	+ 5.3	+ 0.0	+ 16.4	+ 5.2
10 11	+ 6.2	+ 7.1	+ 6.4	+ 0.0	+ 19.7	+ 6.2
12 4	−22.8	−30.1	−102.5	+17.5	−155.4	− 5.3
13 14	+16.3	+17.5	− 18.7	+ 8.2	+ 42.0	− 8.1
15 16	+ 3.3	+ 3.8	+ 13.1	− 6.2	+ 20.2	− 2.9
17 18	+ 3.0	+ 3.4	+ 14.3	− 6.8	+ 20.7	− 3.8
19 20	+ 3.4	+ 4.1	+ 18.2	− 9.2	+ 25.7	− 5.8

Example 17.—Mansard Hip Roof

216. Description.—This roof covers a large hall 200 × 100 ft., between centres of enclosing walls, nearly similar to the armory of the University of Illinois, excepting that its length is increased from 150 to 200 ft., and the trusses are to be entirely of steel, instead of being of wood with diagonals and lower chord of steel. The form of truss is similar. The plan of the entire roof in Fig. 136 *a* shows the arrangement of the trusses, of which there are 5 intermediate trusses, 2 main trusses supporting at their middles the ends of a half truss, and of 2 half hip trusses; 4 half hip trusses; 2 half intermediate trusses; 8 jack trusses of 1/3 span and 8 jack trusses of 1/6 span each. The lower chord is a circular arc, and the panels have equal horizontal lengths. The entire construction of the roof is assumed to be visible internally.

217. Programme.—Type of truss as in Fig. 137; trusses arranged



Plan of Mansard Roof.

as in Fig. 136; span, 100 ft.; rise of upper chord at edge of deck, 17.5 ft., at ridge, 25.0 ft.; rise of lower chord, 10 ft., forming a circular arc; 10 equal horizontal panels; steep side slated on felt and sheathing; deck tinned on sheathing; 1 1/8-inch longleaf pine sheathing; no rafters; steel purlins and trusses; trusses 16 2/3 ft. on centres; location at Columbus, O.; latitude, about 40° north; maximum exposure.

218. Dimensions.—Tan $i = \dfrac{17.5}{10.0} = \tan 60.3°$ for side.

Tan $i = \dfrac{7.5}{40.0} = \tan 10.6°$ for deck.

$$l' = \frac{10.00}{\cos 60.3^\circ} = 20.20 \text{ ft. for side.}$$

$$l' = \frac{10.00}{\cos 10.6^\circ} = 10.17 \text{ ft. for deck.}$$

$A = 20.20 \times 16\ 2/3 = 336.7$ sq. ft. = apex area for side.

$A = 10.17 \times 16\ 2/3 = 169.6$ sq. ft. = apex area for deck.

219. Apex Loads.—

$$\text{Truss} = \frac{100}{25} + \frac{100^2}{12600} = 4.794 \text{ lbs. per horizontal sq. ft.}$$

Snow = 2.5 (40° − 35°) = 12.5 lbs. per horizontal sq. ft.

Wind = 50.00 lbs. per inclined sq. ft. for side.

$$\text{Wind} = \frac{10}{9} \times 10.6^\circ = 11.78 \text{ lbs. per inclined sq. ft. for deck.}$$

$P = 336.7\ (12 + 4.5 + 0 + 4 + 4.794 \cos 60.3^\circ) = 7706$ lbs. = 3.853 tons for side.

$P = 169.6\ (2 + 4.5 + 0 + 4 + 4.794 \cos 10.6^\circ) = 2580$ lbs. = 1.290 tons for deck.

$S = 336.7\ (12.5 \cos 60.3^\circ) = 2084$ lbs. = 1.042 tons for side.

$S = 169.6\ (12.5 \cos 10.6^\circ) = 2082$ lbs. = 1.041 tons for deck.

$W = 336.7 \times 50.00 = 16840$ lbs. = 8.420 tons for side.

$W = 169.6 \times 11.60 = 1964$ lbs. = 0.982 tons for deck.

220. Total Loads on Half Intermediate Truss.—

$$\text{Permanent} = \frac{3.853 + 1.290}{2} + 1.290 \times 3\ 1/2 = 7.09 \text{ tons.}$$

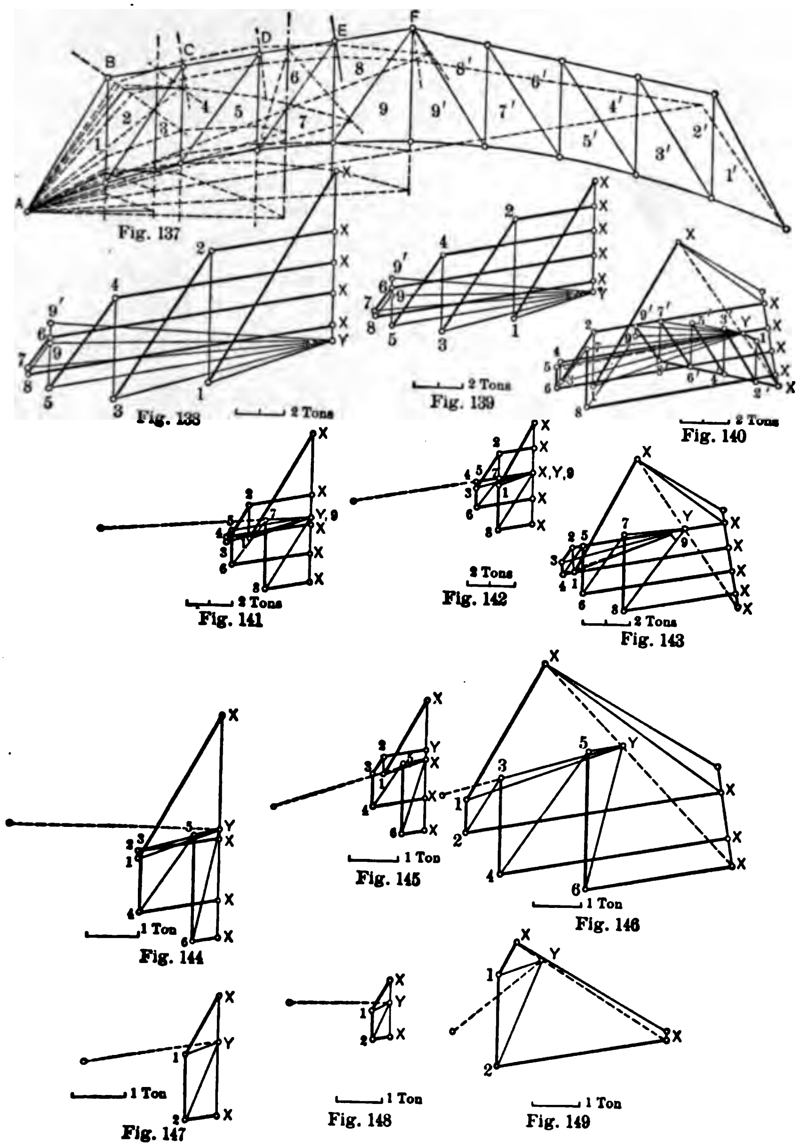
Snow = $1.041 \times 4\ 1/2 = 4.68$ tons.

$$\text{Wind at } B \text{ perpendicular to side} = \frac{8.420}{2} = 4.21 \text{ tons.}$$

$$\text{Wind at } B \text{ perpendicular to deck} = \frac{0.982}{2} = 0.49 \text{ tons.}$$

Wind loads on deck = $0.982 \times 3\ 1/2 = 3.44$ tons.

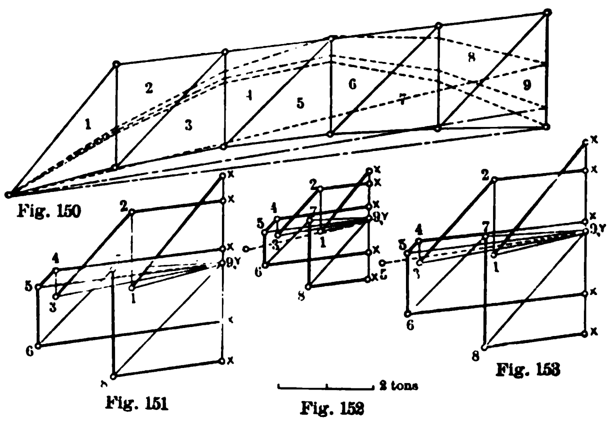
221. Stress Diagrams.—Commence with an intermediate truss, Fig. 137. The permanent stress diagram in Fig. 138, the snow stress diagram in Fig. 139, and the wind stress diagram in Fig. 140



FIGS. 137–149.—Mansard Roof Truss.

present no difficulties. The stress lines for the lower chord evidently radiate from the y-point, since no apex loads are assumed to be supported by the lower chord. The y-point for the wind stress diagram is located by equilibrium polygon, as before.

Taking next the half intermediate truss at the middle of each end of the roof, it is of the same form and has the same loading as the half intermediate truss just considered. But the end A rests on the wall and the inner end F is supported at the middle of the upper chord of the main truss, together with the ends of the two diagonal hip trusses meeting there. No stress exists in the vertical $9\,9'$ or in the member $y\,9$. Figs. 141, 142, and 143 are the corresponding permanent, snow, and wind stress diagrams. The reactions



FIGS. 150–153.—Half Hip Truss

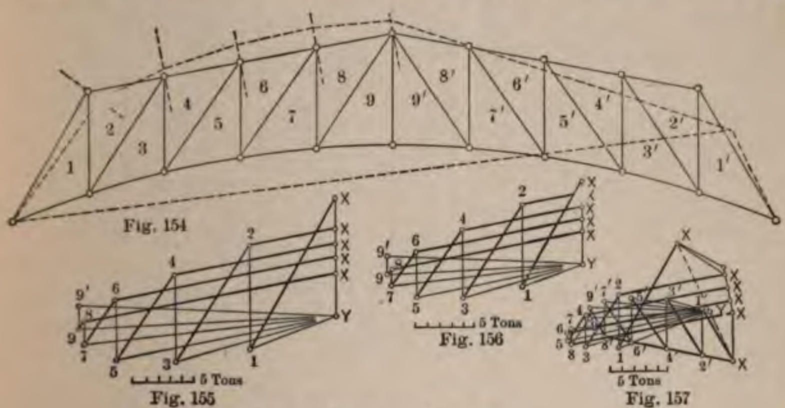
at the ends of the half truss are found by equilibrium polygon. The horizontal component of the wind reaction at the inner end of this truss may be neglected, being small and either resisted by the sheathing of the roof or transmitted through the truss back to the wall.

Similarly, Figs. 144, 145, and 146 are the permanent, snow, and wind stress diagrams for the one-third jack truss; Figs. 147, 148, and 149 are those for the one-sixth jack truss, Fig. 136. The reactions at the upper ends of the jack trusses form the concentrated loads on the diagonal hip trusses, excepting that the horizontal components of the wind reactions are omitted for the reason already given.

Fig. 150 represents the elevation of the half hip truss, in which the horizontal panel length = $10.0\sqrt{2} = 14.14$ ft. The apexes are at the same heights as those corresponding on the intermediate truss, and the apexes of the lower chord lie in an elliptical arc of the same rise as for the half intermediate truss.

The positions of the vertical components of the loads on this truss being determined, the stress diagrams are readily drawn, Figs. 151, 152, and 153 being the permanent, snow, and wind stress diagrams. By the equilibrium polygon must be determined the y-points and the magnitude of the loads transmitted to the middle of the main truss.

Finally, since the additional loads supported at the middle of



Figs. 154–157.—Main Truss.

the main truss have been determined, its stress diagrams can next be drawn. Fig. 154 is the truss diagram, and Figs. 155, 156, and 157 are the respective permanent, snow, and wind diagrams.

The trusses supporting the roof over the Armory of the University of Illinois are of similar form, but were constructed with wooden upper chords and verticals, steel rod diagonals, and lower chord. Their appearance is quite satisfactory, in consideration of their moderate cost.

222. Stress Sheets for Example 17.—

MAIN TRUSSES

Member.	*P*-stress.	*S*-stress.	*W*-stress.	Maximum.	Minimum.
X 1	−14.6	−10.2	−10.0	−34.8	−14.6
X 2	− 7.4	− 5.0	− 9.0	−21.4	− 7.4
X 4	−13.8	−10.3	−12.0	−36.1	−13.8
X 6	−19.0	−14.3	−13.8	−47.1	−19.0
X 8	−21.6	−16.4	−13.8	−51.8	−21.6
Y 1	+ 7.6	+ 5.4	+ 8.0	+21.0	+ 7.6
Y 3	+14.0	+10.5	+10.8	+35.3	+14.0
Y 5	+19.0	+14.2	+12.0	+45.2	+19.0
Y 7	+21.5	+16.0	+11.8	+49.3	+21.5
Y 9	+21.7	+16.5	+10.0	+48.2	+21.7
1 2	+ 8.8	+ 7.0	+ 4.5	+20.3	+ 8.8
3 4	+ 7.5	+ 5.8	+ 2.6	+15.9	+ 7.5
5 6	+ 5.3	+ 3.8	+ 0.8	+ 9.9	+ 5.3
7 8	+ 2.0	+ 1.5	− 1.5	+ 3.5	+ 0.5
9 9′	+ 2.0	+ 1.5	+ 1.0	+ 4.5	+ 2.0
2 3	−11.5	− 9.3	− 5.0	−25.8	−11.5
4 5	− 8.7	− 7.0	− 2.6	−25.0	− 8.7
6 7	− 4.5	− 3.5	+ 0.5	− 8.0	− 4.0
8 9	− 0.8	− 0.5	+ 2.7	− 1.3	+ 1.9

INTERMEDIATE TRUSSES

Member.	*P*-stress.	*S*-stress.	*W*-stress *W*.	*W*-stress *L*.	Maximum.	Minimum.
X 1	−10.4	− 6.9	− 7.3	− 2.2	−24.6	−10.4
X 2	− 5.4	− 3.6	− 7.5	− 1.1	−16.5	− 5.4
X 4	− 9.3	− 6.7	− 9.0	− 2.6	−25.0	− 9.3
X 6	−11.8	− 8.8	− 9.3	− 4.2	−29.9	−11.8
X 8	−12.6	− 9.5	− 8.3	− 5.4	−30.4	−12.6
Y 1	+ 5.6	+ 3.8	+ 6.9	− 0.5	+16.3	+ 5.1
Y 3	+ 9.5	+ 6.9	+ 8.1	+ 1.0	+24.5	+ 9.5
Y 5	+11.8	+ 8.8	+ 8.0	+ 2.6	+28.6	+11.8
Y 7	+12.5	+ 9.4	+ 6.7	+ 3.8	+28.6	+12.5
Y 9	+11.5	+ 8.8	+ 4.7	+ 4.7	+25.0	+11.5
1 2	+ 5.4	+ 4.3	+ 2.2	+ 1.8	+11.9	+ 5.4
3 4	+ 4.0	+ 3.2	+ 0.8	+ 1.9	+ 9.1	+ 4.0
5 6	+ 1.9	+ 1.6	− 1.0	+ 1.8	+ 5.3	+ 0.9
7 8	− 0.3	− 0.2	− 2.5	+ 1.6	+ 1.3	− 3.0
9 9′	+ 1.0	+ 0.7	+ 0.4	+ 0.4	+ 2.1	+ 1.0
2 3	− 7.2	− 5.7	− 2.4	− 2.6	−15.5	− 7.2
4 5	− 4.4	− 3.7	− 0.1	− 2.7	−10.8	− 4.4
6 7	− 1.4	− 1.3	+ 2.1	− 2.3	− 5.0	+ 0.7
8 9	+ 1.6	+ 1.1	+ 3.7	− 1.6	+ 6.4	+ 0.6

HALF TRUSS

Member	*P*-stress.	*S*-stress.	*W*-stress.	Maximum.	Minimum.
X 1	− 4.2	− 3.0	− 5.3	−12.5	− 4.2
X 2	− 2.6	− 1.5	− 6.5	−10.6	− 2.6
X 4	− 3.6	− 2.5	− 7.0	−13.1	− 3.6
X 6	− 3.4	− 2.5	− 6.3	−12.2	− 3.4
X 8	− 2.0	− 1.6	− 4.6	− 8.2	− 2.0
Y 1	+ 2.8	+ 1.6	+ 5.1	+ 9.5	+ 2.8
Y 3	+ 3.7	+ 2.5	+ 5.3	+11.5	+ 3.7
Y 5	+ 3.4	+ 2.5	+ 4.3	+10.2	+ 3.4
Y 7	+ 2.0	+ 1.5	+ 2.3	+ 5.8	+ 2.0
Y 9	+ 0.0	+ 0.0	+ 0.0	+ 0.0	+ 0.0
1 2	+ 1.4	+ 1.3	+ 0.8	+ 3.5	+ 1.4
3 4	+ 0.1	+ 0.3	− 0.7	+ 0.4	− 0.6
5 6	− 2.0	− 1.1	− 2.2	− 5.3	− 2.0
7 8	− 2.8	− 2.2	− 3.2	− 8.2	− 2.8
9 9′	− 0.0	− 0.0	− 0.0	− 0.0	− 0.0
2 3	− 1.8	− 1.8	− 0.5	− 4.1	− 1.8
4 5	− 0.4	− 0.0	+ 1.6	− 0.4	+ 1.2
6 7	+ 2.5	+ 1.6	+ 3.3	+ 7.4	+ 2.5
8 9	+ 3.6	+ 2.9	+ 4.2	+10.7	+ 3.6

HALF HIP TRUSS

Member.	*P*-stress.	*S*-stress.	*W*-stress.	Maximum.	Minimum.
X 1	− 3.1	− 1.6	− 3.1	− 7.8	− 3.1
X 2	− 2.0	− 1.1	− 2.0	− 5.1	− 2.0
X 4	− 3.6	− 2.0	− 3.5	− 9.1	− 3.6
X 6	− 4.0	− 2.3	− 3.8	−10.1	− 4.0
X 8	− 2.4	− 1.3	− 2.1	− 5.8	− 2.4
Y 1	+ 2.0	+ 1.1	+ 2.0	+ 5.1	+ 2.0
Y 3	+ 3.7	+ 2.0	+ 3.6	+ 9.3	+ 3.7
Y 5	+ 4.0	+ 2.3	+ 3.8	+10.1	+ 4.0
Y 7	+ 2.3	+ 1.3	+ 2.2	+ 5.8	+ 2.3
Y 9	+ 0.0	+ 0.0	+ 0.0	+ 0.0	+ 0.0
1 2	+ 1.6	+ 0.9	+ 1.6	+ 4.1	+ 1.6
3 4	+ 0.6	+ 0.4	+ 0.4	+ 1.4	+ 0.6
5 6	− 1.2	− 0.7	− 1.3	− 3.2	− 1.2
7 8	− 2.2	− 1.4	− 2.4	− 6.0	− 2.2
2 3	+ 2.4	+ 1.4	+ 2.4	+ 6.2	+ 2.4
4 5	+ 0.6	+ 0.4	+ 0.4	+ 1.4	+ 0.6
6 7	− 2.3	− 1.4	− 2.4	− 6.1	− 2.3
8 9	− 3.4	− 2.0	− 3.3	− 8.7	− 3.4

ONE-THIRD JACK TRUSS

Member.	*P*-stress.	*S*-stress.	*W*-stress.	Maximum.	Minimum.
X 1	− 3.5	− 1.8	− 3.4	− 8.7	− 3.5
X 2	− 1.8	− 0.9	− 5.6	− 8.3	− 1.8
X 4	− 1.8	− 1.1	− 5.0	− 7.9	− 1.8
X 6	− 0.6	− 0.4	− 3.3	− 4.3	− 0.6
Y 1	+ 1.9	+ 1.0	+ 3.6	+ 6.5	+ 1.9
Y 3	+ 1.8	+ 1.2	+ 2.7	+ 5.7	+ 1.8
Y 5	+ 0.6	+ 0.4	+ 0.8	+ 1.8	+ 0.6
End-rod	+ 2.5	+ 1.7	+ 3.2	+ 7.4	+ 2.5
1 2	+ 0.1	+ 0.3	− 0.7	+ 0.4	− 0.6
3 4	− 1.4	− 0.8	− 2.1	− 4.3	− 1.4
5 6	− 2.3	− 1.6	− 3.0	− 6.9	− 2.3
2 3	+ 0.1	− 0.4	+ 1.5	− 0.3	+ 1.6
4 5	+ 2.1	+ 1.3	+ 3.3	+ 6.7	+ 2.1

ONE-SIXTH JACK TRUSS

Member.	*P*-stress.	*S*-stress.	*W*-stress.	Maximum.	Minimum.
X 1	− 1.5	− 0.7	− 0.8	− 3.0	− 1.5
X 2	− 0.8	− 0.4	− 3.7	− 4.9	− 0.8
Y 1	+ 0.8	+ 0.4	+ 1.1	+ 2.3	+ 0.5
Y 2	+ 1.9	+ 0.9	+ 2.6	+ 5.4	+ 1.9
1 2	− 1.5	− 0.7	− 2.0	− 4.2	− 1.5

EXAMPLE 18.—A COMPOUND TRUSS

223. Description.—For comparison of the appearance and economy of this compound type with those of a simple truss having the same span and rise. This is composed of a primary truss of 4 equal panels, the members of the upper chord being lattice trussed to support intermediate purlins. No rafters. It is to be compared with a simple truss of 8 panels supporting rafters and purlins.

224. Programme.—Truss of type as in Fig. 158; span, 100 ft.; rise, 20 ft.; 4 equal panels in primary truss; material, steel; covering of painted tin on 7/8-inch longleaf pine sheathing; no rafters, steel purlins and trusses; trusses, 16 ft. on centres; 9 equal spaces for intermediate purlins on member of upper chord; location at Pittsburgh; latitude about 40.1° north; medium exposure.

225. Dimensions.—Tan $i = \frac{20}{50} = 0.4000 = \tan 21.8°$.

$$l = \frac{100}{4} = 25.00;\ l' = \frac{25.00}{\cos 21.8°} = 26.925 \text{ ft.}$$

With 9 purlin spaces per member of upper chord, $\frac{26.925}{9} =$ 2.992 ft. O. C.

$A = 2.992 \times 16 = 47.87$ sq. ft. = each purlin area.

226. Purlin Loads.—

Truss $= \frac{100}{25} = \frac{100^2}{12600} = 4.794$ lbs. per horizontal sq. ft.

Snow $= 2.5\ (40.1° - 35°) = 12.75$ lbs. per horizontal sq. ft.

Wind $= \frac{8}{9} \times 21.8° = 18.38$ lbs. per inclined sq. ft.

$P = 47.87\ (2 + 4 + 0 + 4 + 4.794 \cos 21.8°) = 692$ lbs.

$S = 47.87\ (12.75 \cos 21.8°) = 567$ lbs.

$W = 47.87 \times 18.38 = 880$ lbs.

227. Total Loads on Member of Upper Chord.—

Permanent = 692 × 8 = 5538 lbs. = 2.77 tons.

Snow = 567 × 8 = 4536 lbs. = 2.27 tons.

Wind = 880 × 8 = 7040 lbs. = 3.52 tons.

228. Total Loads on Half Primary Truss.—

Permanent = 692 × 13 1/2 = 9342 lbs. = 4.67 tons.

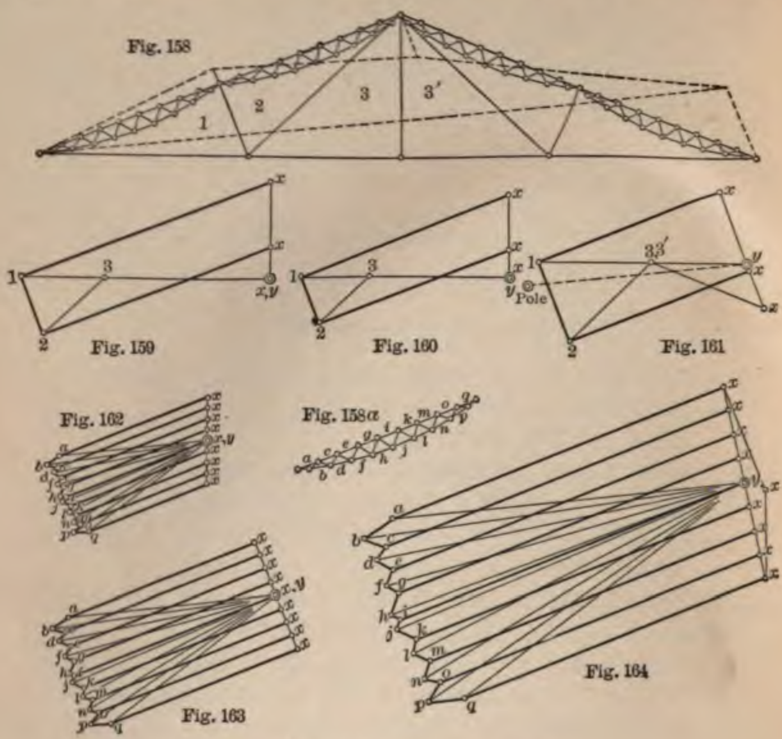
Snow = 567 × 13 1/2 = 7655 lbs. = 3.83 tons.

Wind = 880 × 13 1/2 = 11880 lbs. = 5.94 tons.

229. Stress Diagrams.—Figs. 159, 160, and 161 are the permanent, snow, and wind stress diagrams for the primary Fink truss of 4 panels.

The lattice trussed member, Fig. 158 *a*, of the primary truss, is to be treated as a separate truss, supported at its ends by the primary truss and carrying 8 purlins and their loads. Fig. 162 is its *P* stress diagram. The snow stresses may be found by the porportion: 2.77 : 2.27 : : *P* stress : *S* stress on the same member. Fig. 163 is its *W* stress diagram. The panels of this lattice truss are lettered *a* to *q* from the lower end. The *P*, *S*, and *W* stresses in the lattice members are entered on the stress sheet as shown. Then the maximum total lattice stress occurring on any member of upper chord is entered on the stress sheet from primary truss, since the member must safely resist such stress in addition to that due to the primary truss.

Or the maximum stresses in the members of the lattice truss may be found by a single stress diagram, as in Fig. 164. Here the load



FIGS. 158–164.—Compound Truss.

line = (P load + S load) $\cos i + W$ = (2.77 + 2.27) cos 21.8° + 3.52 = 8.20 tons.

230. Stress Sheets for Example 18.—

PRIMARY TRUSS

Member.	*P*-stress.	*S*-stress.	*W*-stress.	Lattice.	Maximum.	Minimum.
X 1	−12.5	−10.4	− 9.2	−15.8	−47.9	−17.5
X 2	−11.4	− 9.4	− 9.2	−15.8	−45.8	−16.5
Y 1	+11.6	+ 9.4	+ 9.2	+ 0.0	+31.1	+11.6
Y 3	+ 7.7	+ 6.4	+ 4.5	+ 0.0	+18.6	+ 7.7
1 2	− 2.9	− 2.3	− 4.0	− 0.0	− 9.2	− 2.9
3 4	+ 3.9	+ 3.2	+ 5.4	+ 0.0	+12.5	+ 3.9

TRUSSED MEMBER OF UPPER CHORD

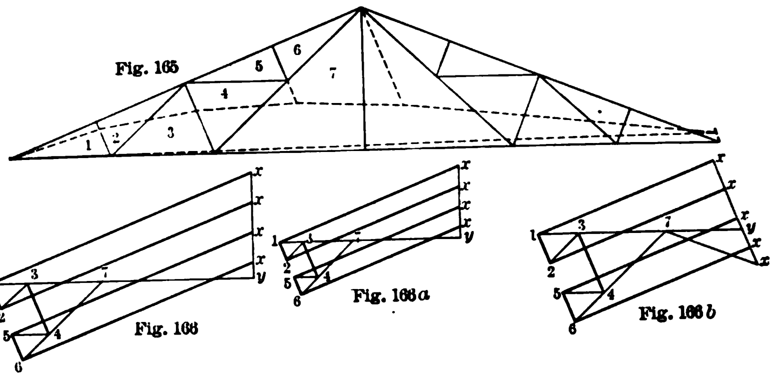
Member.	*P*-stress.	*S*-stress.	*W*-stress.	Maximum.
X a	− 4.7	− 3.9	− 6.0	−14.6
X c	− 5.0	− 4.1	− 6.6	−15.7
X e	− 5.0	− 4.1	− 6.7	−15.8
X g	− 4.9	− 4.0	− 6.7	−15.6
X i	− 4.9	− 4.0	− 6.7	−15.6
X k	− 4.8	− 3.9	− 6.7	−15.4
X m	− 4.6	− 3.8	− 6.7	−15.1
X o	− 4.4	− 3.6	− 6.5	−14.5
X q	− 4.1	− 3.4	− 6.2	−13.7
Y b	+ 4.4	+ 3.6	+ 6.3	+14.3
Y d	+ 4.9	+ 4.0	+ 7.0	+15.9
Y f	+ 4.9	+ 4.0	+ 7.0	+15.9
Y h	+ 4.9	+ 4.0	+ 6.9	+15.8
Y j	+ 4.9	+ 4.0	+ 6.9	15.8
Y l	+ 5.0	+ 4.1	+ 6.9	16.0
Y n	+ 5.0	+ 4.1	+ 7.0	16.1
Y p	+ 5.1	+ 4.2	+ 6.9	+16.2
Y q	+ 4.7	+ 3.9	+ 6.4	15.0

Member.	*P*-stress.	*S*-stress.	*W*-stress.	Maximum.
a b	− 0.6	− 0.5	− 0.8	− 1.9
c d	− 0.3	− 0.2	− 0.4	− 0.9
e f	− 0.3	− 0.2	− 0.3	− 0.8
g h	− 0.2	− 0.2	− 0.3	− 0.7
i j	− 0.2	− 0.2	− 0.3	− 0.7
k l	− 0.2	− 0.2	− 0.3	− 0.7
m n	− 0.3	− 0.2	− 0.2	− 0.7
o p	− 0.3	− 0.2	− 0.3	− 0.8
b c	+ 0.2	+ 0.2	+ 0.3	+ 0.7
d e	+ 0.2	+ 0.2	+ 0.3	+ 0.7
f g	+ 0.2	+ 0.2	+ 0.3	+ 0.7
h i	+ 0.2	+ 0.2	+ 0.3	+ 0.7
j k	+ 0.2	+ 0.2	+ 0.3	+ 0.7
l m	+ 0.2	+ 0.2	+ 0.3	+ 0.7
n o	+ 0.2	+ 0.2	+ 0.5	+ 0.9
p q	+ 0.5	+ 0.4	+ 0.6	+ 1.5

EXAMPLE 19.—A SIMPLE FINK TRUSS

231. Description.—For comparison of stresses, efficiency, and economy with the compound truss of Example 18.

232. Programme.—Truss of type as in Fig. 165; span, 100 ft.; rise, 20 ft.; 8 equal panels; material, steel; covering of painted tin



FIGS. 165–166 *b*.—Fink Truss for same Roof.

on 7/8-inch longleaf pine sheathing; steel rafters, purlins, and truss; trusses, 16 ft. on centres; location at Pittsburgh; latitude about 40.1° north; medium exposure.

233. Dimensions.—

$$\text{Tan } i = \frac{20}{50} = 0.4000 = \tan 21.8°.$$

$$l = \frac{100}{25} = 12.5 \text{ ft.}; \quad l' = \frac{12.5}{\cos 21.8°} = 13.46 \text{ ft.}$$

$A = 13.46 \times 16.00 = 215.4$ sq. ft. = apex area.

234. Apex Loads.—

Truss = 4.794 lbs. per horizontal sq. ft., as before.

Snow = 12.75 lbs. per horizontal sq. ft., as before.

Wind = 18.38 lbs. per inclined sq. ft., as before.

$P = 215.4\ (2+4+4+3+4.794 \cos 21.8°) = 3759$ lbs. $= 1.880$ tons.

$S = 215.4\ (12.75 \cos 21.8°) = 2550$ lbs. $= 1.275$ tons.

$W = 215.4 \times 18.38 = 3959$ lbs. $= 1.980$ tons.

235. Total Loads on Half Truss.—

Permanent $= 1.880 \times 3\ 1/2 = 6.58$ tons.

Snow $= 1.275 \times 3\ 1/2 = 4.46$ tons.

Wind $= 1.980 \times 3\ 1/2 = 6.93$ tons.

236. Stress Diagrams.—The permanent, stress, and wind diagrams are given in Figs. 166, 166 *a*, and 166 *b*.

237. Stress Sheet for Example 19.—

Member.	*P*-stress.	*S*-stress.	*W*-stress.	Maximum.	Minimum.
X 1	−17.8	−12.1	−11.5	−41.4	−17.8
X 2	−17.1	−11.6	−11.5	−40.2	−17.1
X 5	−16.4	−11.1	−11.5	−39.0	−16.4
X 6	−15.7	−10.6	−11.5.	−37.8	−15.7
Y 1	+16.5	+11.2	+12.4	+40.1	+16.5
Y 3	+14.3	+ 9.7	+ 9.8	+33.8	+14.3
Y 7	+ 9.6	+ 6.5	+ 4.4	+20.5	+ 9.6
1 2	− 1.7	− 1.2	− 2.0	− 4.9	− 1.7
3 4	− 3.5	− 2.4	− 4.0	− 9.9	− 3.5
5 6	− 1.7	− 1.2	− 2.0	− 4.9	− 1.7
2 3	+ 2.3	+ 1.6	+ 2.6	+ 6.5	+ 2.3
4 5	+ 2.3	+ 1.6	+ 2.6	+ 6.5	+ 2.3
4 7	+ 4.7	+ 3.2	+ 5.4	+13.3	+ 4.7
6 7	+ 7.0	+ 4.7	+ 8.0	+19.7	+ 7.0

Example 20.—Mansard Hip Roof with Ceiling

238. Description.—This roof is designed for a building of the same dimensions as in Example 17, being intended to cover a large hall in a fireproof structure, with a horizontal ceiling divided into square panels by beams 12.5 ft. on centres. Therefore the trusses are not visible in the interior. The ceiling and the surface of the roof are each composed of concrete slabs 3 inches thick, the roof slab being cement plastered on top and the ceiling slab hard plastered on its visible surface. The steel trusses do not require to be fireproofed, being enclosed between the fireproof roof and ceiling.

239. Programme.—Truss of type as in Fig. 167; span, 100 ft.; rise of upper chord at ridge, 20 ft.; at edge of deck, 15 ft.; 8 equal panels; materials, steel and reënforced concrete, supported by steel purlins, beams, and trusses; roof covered by 1-inch cement plaster; deck covered by felt and gravel cemented to plastering; side covered by tiles on ruberoid felt on plastering; ceiling of 3-inch slab of reenforced concrete plastered beneath, supported by lower chord of truss; no fireproofing on trusses; location at Toronto; latitude, about 43 2/3° north; maximum exposure.

240. Dimensions.—

$$\text{Tan } i = \frac{15.0}{12.5} = \tan 50.2° \text{ for side.}$$

$$\text{Tan } i = \frac{5.0}{37.5} = \tan 6.6° \text{ for deck.}$$

$$l = \frac{100}{8} = 12.5 \text{ ft.}$$

$$l' = \frac{12.5}{\cos 50.2°} = 19.54 \text{ ft. for side; } l'' = \frac{12.5}{\cos 6.6°} = 12.59 \text{ ft. for deck.}$$

$A = 19.54 \times 12.5 = 244.25$ sq. ft. = apex area for side.
$A = 12.59 \times 12.5 = 157.38$ sq. ft. = apex area for deck.
$A = 12.50 \times 12.5 = 156.25$ sq. ft. = apex area for ceiling.

241. Apex Loads.—

$$\text{Truss} = \frac{100}{25} + \frac{100^2}{12600} = 4.794 \text{ lbs. per horizontal sq. ft.}$$

Snow = 2.5 (43 2/3° − 35°) = 21.67 lbs. per horizontal sq. ft.

$$\text{Wind} = \frac{10}{9} \times 6.6° = 7.33 \text{ lbs. per inclined sq. ft. for deck.}$$

Wind = 50.00 lbs. per inclined sq. ft. for side.

Tiles 11 lbs., plaster 10 lbs., per sq. ft.; concrete 150 lbs. per cu. ft.

$P = 244.25\ (11 + 10 + 37.5 + 4 + 4.794 \cos 50.2°) = 15794$ lbs. = 7.897 tons.

$P = 157.38\ (6 + 10 + 37.5 + 4 + 4.794 \cos 6.6°) = 9754$ lbs. = 4.877 tons.

$P = 156.25\ (10 + 37.5 + 4) = 8047$ lbs. = 4.024 tons.
$S = 244.25\ (21.67 \cos 50.2°) = 3385$ lbs. = 1.693 tons.
$S = 157.38\ (21.67 \cos 6.6°) = 3387$ lbs. = 1.694 tons.
$W = 244.25 \times 50.00 = 12213$ lbs. = 6.107 tons.
$W = 157.38 \times 7.33 = 1154$ lbs. = 0.577 tons.

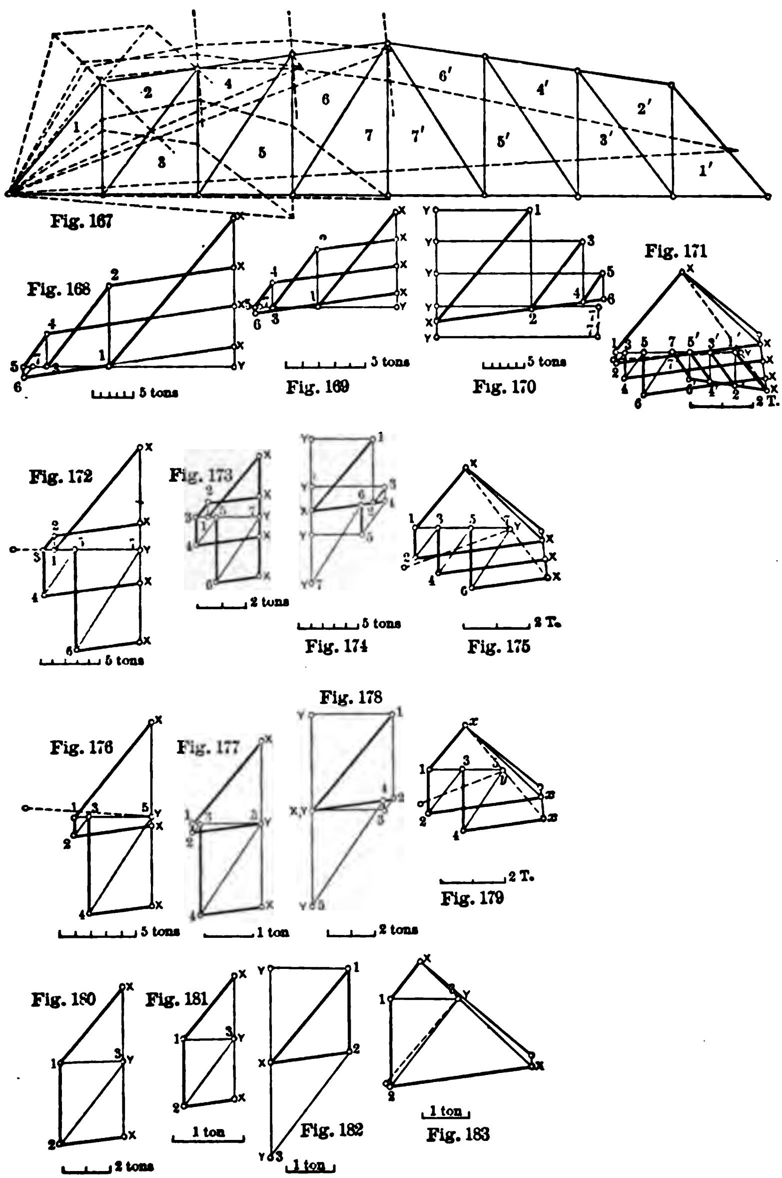
242. Total Loads on Half Truss.—

$$\text{Permanent} = \frac{7.897 + 4.877}{2} + 4.877 \times 2\,1/2 = 18.58 \text{ tons. On U.C.}$$

Permanent = 4.024 × 3 1/2 = 14.08 tons. On L. Chord.
Snow = 1.694 × 3 1/2 = 5.93 tons.

$$\text{Wind} = \frac{6.107}{2} = 3.05 \text{ tons perpendicular to side.}$$

Wind = 0.577 × 3 1/2 = 1.73 tons perpendicular to deck.
Resultant of wind loads is to be found graphically.



FIGS. 167–183.—Mansard Roof Truss with Ceiling.

243. Stress Diagrams.—On account of the construction of roof and ceiling in reënforced concrete, the loads to be supported are much greater than in Example 17; the lower chord is horizontal and supports a fireproof ceiling; yet the stress diagrams are drawn nearly as in that example.

Fig. 167 represents an intermediate truss; Figs. 168, 169, 170, and 171 are the stress diagrams for *P*, *S*, ceiling, and *W* loads.

For the intermediate half-truss at each end of the roof, the left half of the truss diagram in Fig. 167 is employed, the end reactions being obtained by the equilibrium polygon, and Figs. 172, 173, 174, and 175 are the *P*, *S*, ceiling, and *W* stress diagrams.

The *P*, *S*, ceiling, and *W* stress diagrams for the three-eighth jack trusses are similarly drawn in Figs. 176, 177, 178, and 179. Figs. 180, 181, 182, and 183 are the stress diagrams for the one-fourth jack trusses. Fig. 184 is the truss diagram for the half-hip or diagonal trusses at the angles of the building. Since these trusses are unable to resist any forces acting at right angles to their middle vertical planes, only the vertical components of the wind reactions at ends of jack trusses supported by them are here regarded as loads on these hip trusses; the horizontal components are resisted by the principals of the other jack trusses attached at the same points of the hip truss and by the sheathing of the roof.

The *P*, *S*, *C*, and *W* stress diagrams are given in Figs. 185, 186, 187, and 188.

Fig. 189 is the truss diagram for the main trusses, each of which at its middle supports the inner ends of a half-truss and two hip trusses, therefore having these additional loads at the middle apexes of upper and lower chords. Otherwise its loads are the same as for an intermediate truss. The *P*, *S*, *C*, and *W* stress diagrams are shown in Figs. 190, 191, 192, and 193.

About two-thirds of the horizontal component of the wind pressure on the end slope of the hip roof of the building would be supported by the end wall of the building; one-third would be carried to the middle apex of the upper chord of the main truss, then producing equal horizontal compression in each ridge purlin between the two main trusses, being finally resisted by the two hip trusses and the half-truss meeting at the corresponding apex of the main truss at leeward end of the roof. Hence no longitudinal truss is required to connect the middle apexes of the main and intermediate trusses.

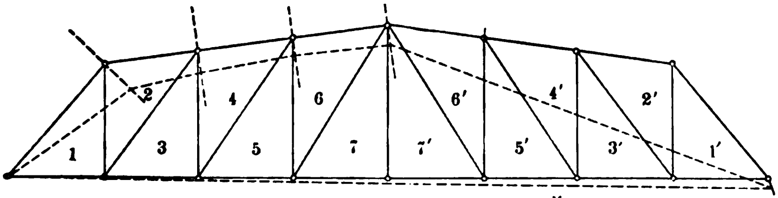


Fig. 189

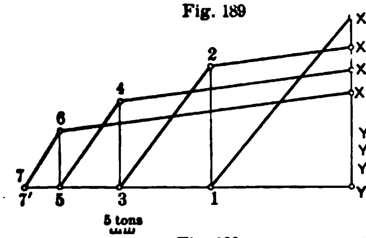


Fig. 190

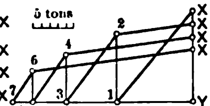


Fig. 191

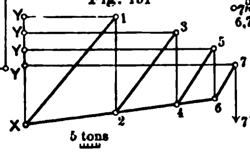


Fig. 192

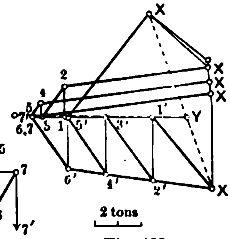


Fig. 193

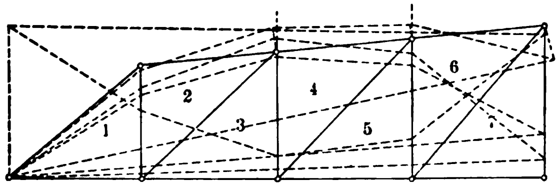


Fig. 184

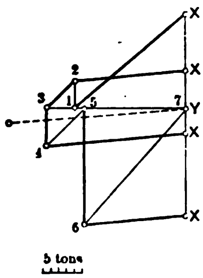


Fig. 185

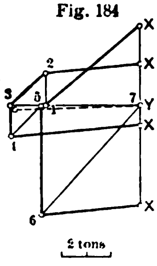


Fig. 186

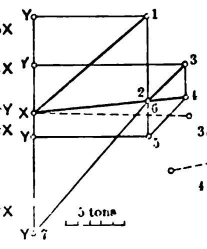


Fig. 187

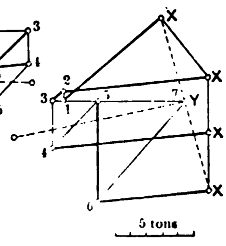


Fig. 188

Figs. 184-193.—Mansard Roof Truss with Ceiling.

244. Stress Sheets for Example 20.—

INTERMEDIATE TRUSS

Member.	*P*-stress.	*S*-stress.	*W*-stress *W*.	*W*-stress *L*.	*C*-stress.	Maximum.
X 1	−24.2	− 8.0	− 3.3	− 1.6	−18.4	−53.9
X 2	−15.7	− 5.0	− 4.5	− 1.0	−11.8	−37.0
X 4	−23.3	− 7.8	− 4.4	− 1.8	−18.3	−53.7
X 6	−26.2	− 8.9	− 3.8	− 2.5	−21.0	−59.9
Y 1	+15.5	+ 5.0	+ 3.8	+ 0.2	+11.7	+36.0
Y 3	+23.1	+ 8.8	+ 3.6	+ 1.0	+18.2	+53.7
Y 5	+26.0	+ 8.9	+ 3.0	+ 1.6	+20.7	+58.6
Y 7	+25.3	+ 8.6	+ 2.1	+ 2.1	+20.3	+56.3
1 2	+10.2	+ 3.7	− 0.3	+ 1.1	+12.5	+27.5
3 4	+ 4.3	+ 1.6	− 0.9	+ 1.0	+ 7.7	+14.6
5 6	− 1.0	− 0.3	− 1.4	+ 0.9	+ 3.4	+ 3.3
7 7′	− 0.0	− 0.0	− 0.0	+ 0.0	+ 4.0	+ 4.0
2 3	−12.7	− 4.6	+ 0.4	− 0.4	−10.7	−28.4
4 5	− 5.2	− 2.0	+ 1.1	− 1.1	− 4.5	−12.8
6 7	+ 1.3	+ 0.4	+ 1.6	− 1.6	+ 0.8	+ 4.1

THREE-EIGHTH TRUSS

Member.	*P*-stress.	*S*-stress.	*W*-stress.	*C*-stress.	Maximum.
X 1	− 7.8	− 2.3	− 1.7	− 5.3	−17.1
X 2	− 5.0	− 1.5	− 3.5	− 3.4	−13.4
X 4	− 4.2	− 1.3	− 2.6	− 3.0	−11.1
Y 1	+ 5.0	+ 1.5	+ 2.3	+ 3.4	+12.2
Y 3	+ 4.2	+ 1.3	+ 1.3	+ 3.0	+ 9.8
Y 5	+ 0.0	+ 0.0	+ 0.0	+ 0.0	+ 0.0
1 2	− 1.0	− 0.2	− 1.4	+ 3.5	− 2.6
3 4	− 6.0	− 1.9	− 1.8	− 0.5	−10.1
2 3	+ 1.3	+ 0.3	+ 1.7	+ 0.6	+ 3.9
4 5	+ 7.3	+ 2.3	+ 2.2	+ 5.5	+17.1

ONE-FOURTH JACK TRUSS

Member.	*P*-stress.	*S*-stress.	*W*-stress.	*C*-stress	Maximum.
X 1	− 4.2	− 1.2	− 1.0	− 2.5	− 8.9
X 2	− 2.7	− 0.8	− 3.1	− 1.7	− 8.3
Y 1	+ 2.7	+ 0.8	+ 1.5	+ 1.6	+ 6.6
Y 3	+ 0.0	+ 0.0	+ 0.0	+ 0.0	+ 0.0
1 2	− 3.5	− 1.0	+ 1.9	− 1.7	− 6.2
2 3	+ 4.5	+ 1.2	+ 2.4	+ 2.8	+10.9

HALF TRUSS

Member	*P*-stress.	*S*-stress.	*W*-stress.	*C*-stress.	Maximum.
X 1	−11.0	− 3.3	− 2.4	− 8.0	−24.7
X 2	− 7.2	− 2.2	− 3.9	− 5.2	−18.5
X 4	− 9.0	− 2.6	− 3.3	− 6.2	−21.1
X 6	− 5.5	− 1.8	− 2.4	− 4.2	−13.9
Y 1	+ 7.1	+ 2.1	+ 2.9	+ 5.1	+17.2
Y 3	+ 8.0	+ 2.6	+ 2.2	+ 6.1	+18.9
Y 5	+ 5.4	+ 1.7	+ 1.2	+ 4.1	+12.4
Y 7	+ 0.0	+ 0.0	+ 0.0	+ 0.0	+ 0.0
1 2	+ 1.2	+ 0.6	+ 1.0	+ 5.5	+ 7.3
3 4	− 3.9	− 1.2	− 1.4	+ 1.2	− 5.3
5 6	− 8.5	− 2.7	− 1.9	− 2.5	−15.6
7 7′	− 0.0	− 0.0	− 0.0	− 0.0	− 0.0
2 3	− 1.5	− 1.7	− 1.2	− 1.8	− 5.0
4 5	+ 4.7	+ 1.5	+ 1.7	+ 3.5	+ 9.9
6 7	+10.0	+ 3.3	+ 2.2	+ 7.9	+20.1

HIP HALF TRUSS

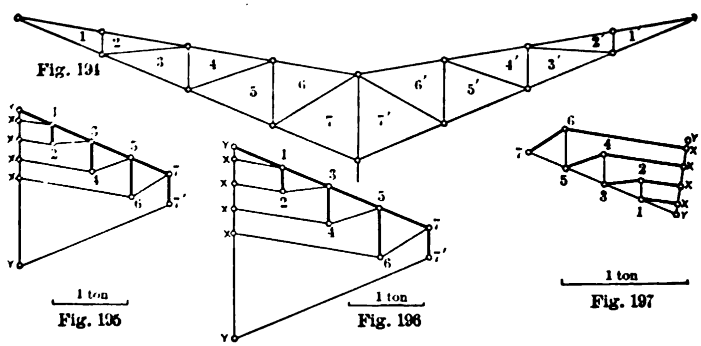
Member.	*P*-stress.	*S*-stress.	*W*-stress.	*C*-stress.	Maximum.
X 1	−18.0	− 5.0	− 3.9	−12.2	−39.1
X 2	−13.6	− 3.8	− 4.5	− 9.3	−31.2
X 4	−17.1	− 5.2	− 4.8	−12.5	−39.6
X 6	−12.3	− 3.9	− 3.4	− 9.4	−29.0
Y 1	+13.5	+ 3.8	+ 3.7	+ 9.3	+30.3
Y 3	+17.0	+ 5.2	+ 4.0	+12.3	+38.5
Y 5	+12.3	+ 3.8	+ 2.5	+ 9.3	+27.9
Y 7	+ 0.0	+ 0.0	+ 0.0	+ 0.0	+ 0.0
1 2	+ 3.4	+ 1.3	+ 0.3	+ 7.0	+12.0
3 4	− 5.0	− 1.4	− 1.5	+ 2.7	− 5.2
5 6	−14.8	− 4.6	− 2.2	+ 3.0	−18.6
7 7′	− 0.0	− 0.0	− 0.0	− 0.0	− 0.0
2 3	− 4.8	− 1.9	− 0.4	− 4.3	−11.4
4 5	+ 6.9	+ 2.0	+ 2.1	+ 4.5	+15.5
6 7	+19.2	+ 6.0	+ 4.1	+14.6	+43.9

MAIN TRUSS

Member.	P-stress.	S-stress.	W-stress.	C-stress.	Maximum.
X 1	−47.0	−15.0	− 5.7	−35.7	−103.4
X 2	−33.5	− 9.7	− 6.1	− 2.3	− 51.6
X 4	−49.6	−16.2	− 7.1	−37.6	−110.5
X 6	−62.2	−20.3	− 7.6	−47.5	−137.6
Y 1	+30.0	+ 9.6	+ 5.2	+22.7	+ 67.5
Y 3	+49.1	+16.0	+ 6.1	+36.3	+110.9
Y 5	+61.6	+20.1	+ 6.5	+46.8	+135.0
Y 7	+68.0	+22.5	+ 6.4	+52.3	+149.2
1 2	+25.7	+ 8.6	+ 1.3	+24.2	+ 61.2
3 4	+18.4	+ 6.1	+ 0.6	+18.4	+ 45.3
5 6	+12.0	+ 3.8	− 0.1	+13.0	+ 31.0
7 7′	+ 0.0	+ 0.0	+ 0.0	+ 0.0	+ 0.0
2 3	−32.1	−10.8	− 1.0	−25.0	− 71.2
4 5	−22.2	− 7.4	− 0.7	−17.2	− 49.7
6 7	−14.1	− 4.5	+ 0.1	−10.3	− 31.5

EXAMPLE 21.—DOUBLE CANTILEVER TRUSS

245. Description.—This type of roof is now frequently employed as an economical substitute for the former great railway-train halls



FIGS. 194–197.—Roof Truss over Platform.

with roofs of wide span, and it is particularly useful where heavy snow falls do not occur. This roof covers a double platform between two tracks and partly extends over the trains to shelter the passengers from rain, but an open space over the track is left for light and ventilation. Each transverse roof truss is fixed to a single column strongly anchored by a concrete block, etc. The apexes of the

upper chord are usually connected by purlins, on which is fastened the wooden sheathing or corrugated steel roofing. Rafters are not necessary. The covering is best made of felt, asphalt, and gravel, or of painted tin on wooden sheathing, forming a central valley with leaders down to the drain at each column. The wind pressure here acts upward on the under side of the sheathing of the outer side of the roof only, unless the roof is protected by an outer wall or an adjacent building. The stability of such columns will be considered later.

246. Programme.—Truss of type as in Fig. 194; span, 24 ft.; drop of upper chord, 2 ft., of lower chord 5 ft.; 8 panels; materials, steel purlins, truss, and column; trusses 15 ft. on centres; covering of painted tin on 7/8-inch longleaf pine sheathing; location at Pittsburgh, latitude about 40 1/2° north; ordinary exposure.

247. Dimensions.—

$$\text{Tan } i = \frac{2}{12} = 0.1667 = \tan 9.5° \text{ reversed.}$$

$$l = \frac{24}{8} = 3.00 \text{ ft.}; \quad l' = \frac{3.00}{\cos 9.5°} = 3.042 \text{ ft.}$$

$$A = 3.042 \times 15.00 = 45.62 \text{ sq. ft.} = \text{apex area.}$$

248. Apex Loads.—

$$\text{Truss} = \frac{24}{25} + \frac{24^2}{12600} = 1.006 \text{ lbs. per horizontal sq. ft.}$$

$$\text{Snow} = 2.5\ (40.5°) = 13.75 \text{ lbs. per horizontal sq. ft.}$$

$$\text{Wind} = \frac{2}{3} = 9.5° = 6.3 \text{ lbs. per inclined sq. ft.}$$

$$P = 45.62\,(2 + 4 + 0 + 4 + 1.006 \cos 9.5°) = 501 \text{ lbs.} = 0.251 \text{ ton.}$$

$$S = 45.62\,(13.75 \cos 9.5°) = 619 \text{ lbs.} = 0.310 \text{ ton.}$$

$$W = 45.62 \times 6.3 = 287 \text{ lbs.} = 0.144 \text{ ton.}$$

249. Total Loads on Half Truss.—

$$\text{Permanent} = 0.251 \times 4 = 1.004 \text{ tons.}$$

$$\text{Snow} = 0.310 \times 4 = 1.240 \text{ tons.}$$

$$\text{Wind} = 0.144 \times 4 = 0.576 \text{ ton.}$$

250. Stress Diagrams.—The truss diagram is shown in Fig. 194; Figs. 195, 196, and 197 are the *P*, *S*, and *W* stress diagrams. Note that the wind loads act upward on the truss and are so laid off in Fig. 197.

251. Stress Sheet for Example 21.—

Member.	*P*-stress.	*S*-stress.	*W*-stress.	Maximum.
X 1	+ 0.5	+ 0.7	− 0.3	+ 1.2
X 2	+ 0.5	+ 0.7	− 0.3	+ 1.2
X 4	+ 1.3	+ 1.3	− 0.7	+ 2.3
X 6	+ 1.5	+ 2.0	− 1.0	+ 3.5
Y 1	− 0.5	− 0.7	+ 0.3	− 1.2
Y 3	− 1.1	− 1.4	+ 0.6	− 2.5
Y 5	− 1.6	− 2.1	− 1.0	− 3.7
Y 7	− 2.2	− 2.8	− 1.3	− 5.0
1 2	− 0.3	− 0.3	+ 0.1	− 0.6
3 4	− 0.4	− 0.5	+ 0.2	− 0.9
5 6	− 0.5	− 0.7	+ 0.3	− 1.2
7 7′	− 0.4	− 0.4	+ 0.0	− 0.8
2 3	+ 0.5	+ 0.6	− 0.3	+ 1.1
4 5	+ 0.6	+ 0.7	− 0.3	+ 1.3
6 7	+ 0.6	+ 0.8	− 0.4	+ 1.4

Example 22.—Cantilever and Skylight Truss

252. Description.—This truss consists of two cantilever trusses connected over the middle span and supporting a large central gable skylight (Fig. 198). It is supported by two stable walls or anchored columns, whose stability will be considered later. The steeper gable over the middle span is a glazed skylight, which abundantly lights the middle portion of the building, even if the truss be supported by walls. External gutters at its base discharge on the slopes of the lower sides, and internal drip gutters may be similarly arranged, avoiding the use of internal leaders to the drains. This form of skylight is common in Germany, being used instead of the sawtooth roof, avoiding all the defects of that type. To make the roof truss entirely stable, it is necessary to connect the cantilever trusses by a middle truss with parallel chords beneath the skylight. The reactions at the ends of the skylight then become additional loads at its points of attachment to the main truss, its stresses being separately determined.

253. Programme.—Truss of type as in Fig. 198; total span, 200 ft.; rise of upper chord produced to centre, 40 ft.; rise of skylight 20 ft. additional, making a total rise of 60 ft. at ridge; scissors truss for skylight; material, steel; covered with painted tin on 1 3/8

white pine sheathing; no rafters; steel purlins and truss; steel T-bars for skylight to receive glass; trusses 20 ft. on centres; location at Portland, Me.; latitude about 43 2/3° north; medium exposure.

254. Dimensions.—

$$\text{Tan } i = \frac{20}{50} = 0.4000 = \tan 21.8° \text{ for tinned roof.}$$

$$\text{Tan } i = \frac{30}{25} = 1.2000 = \tan 50.2° \text{ for glazed roof.}$$

$$l = \frac{200}{16} = 12.5 \text{ ft.}; \; l' = \frac{12.5}{\cos 21.8°} = 13.47 \text{ ft. for tinned}$$

roof.

$$l'' = \frac{12.5}{\cos 50.2°} = 19.54 \text{ ft. for glazed roof.}$$

$A = 13.47 \times 20.00 = 269.4$ sq. ft. = apex area for tinned roof.
$A = 19.54 \times 20.00 = 390.8$ sq. ft. = apex area for glazed roof.

255. Apex Loads.—It will probably suffice to assume weight of truss for 100 ft. span.

Truss = 4,794 lbs. per horizontal sq. ft., as before.
Snow = 2.5 (43.7° − 35°) = 21.67 lbs. per horizontal sq. ft.

$$\text{Wind} = \frac{8}{9} \times 21.8° = 19.38 \text{ lbs. per inclined sq. ft.}$$

$P = 269.4\,(2 + 5.5 + 0 + 4 + 4.794 \cos 21.8°) = 4297$ lbs. = 2.149 Tons for tin roof.

$P = 390.8\,(5 + 0 + 4 + 3 + 4.794 \cos 50.2°) = 5889$ lbs. = 2.945 Tons, glass roof.

$S = 269.4\,(21.67 \cos 21.8°) = 5420$ lbs. = 2.710 Tons., tin roof.
$S = 390.8\,(21.67 \cos 50.2°) = 5416$ lbs. = 2.708 Tons., glass roof.
$W = 269.4 \times 19.38 = 5221$ lbs. = 2.611 Tons., tin.
$W = 390.8 \times 40.00 = 15632$ lbs. = 7.816 tons for glass roof.

256. Total Loads on Half Truss.—

Permanent = $2.149 \times 6 + 2.945 \times 2 = 18.78$ tons.
Snow = $2.710 \times 6 + 2.708 \times 2 = 21.68$ tons.
Wind = $2.611 \times 6 = 15.67$ tons on tinned roof.
Wind = $7.816 \times 2 = 15.63$ tons on glazed roof.

257. Stress Diagrams.—The stress diagrams are first commenced at the outer end of left cantilever; next at middle apex of skylight, for P and S diagrams, since only the stress diagrams for one-half the truss are required. The stress in the member Y 10 is here found in the horizontal upper chord of the middle portion of the truss as

tension, and as compression in the corresponding horizontal lower chord. Figs. 199 and 200 are the P and S stress diagrams.

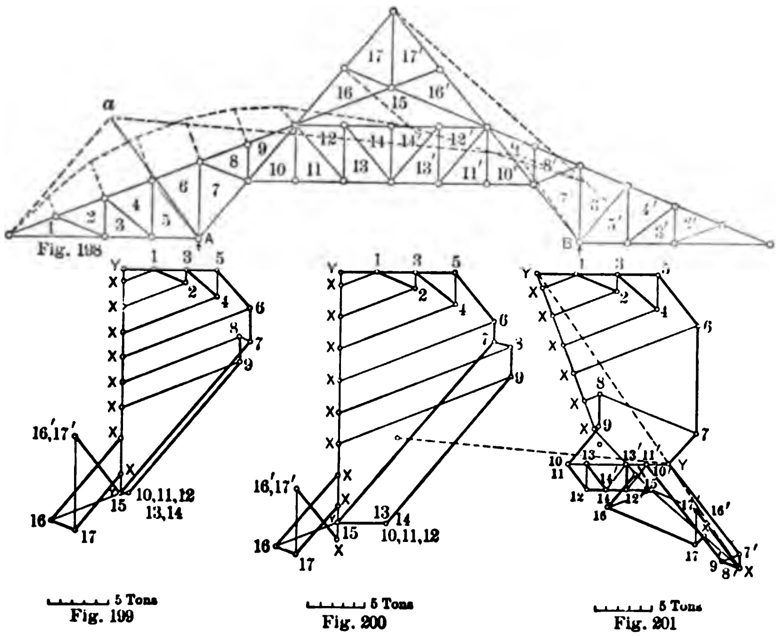
The W reactions at the supports A and B are found by the equilibrium polygon, commencing at a on a line drawn through support A and parallel to the resultant of wind loads in Fig. 201, ending on a similar parallel drawn through support B. The W stress diagram is then commenced at the free end of left cantilever and continued as far as the middle of the truss, beginning again at support B and thus completing the stress diagram in Fig. 201.

258. Stress Sheet for Example 22.—

Member.	P-stress.	S-stress.	W-stress W.	W-stress L.	Maximum.	Minimum.
X 1	+ 2.9	+ 3.7	+ 3.2	+ 0.0	+ 9.8	+ 2.9
X 2	+ 5.8	+ 7.3	+ 5.9	+ 0.0	+19.0	+ 5.8
X 4	+ 8.8	+11.0	+ 8.7	+ 0.0	+28.5	+ 8.8
X 6	+11.7	+14.6	+11.3	+ 0.0	+37.6	+11.7
X 8	+11.1	+16.5	+ 1.6	− 1.3	+29.1	+10.8
X 9	+11.1	+16.5	+ 0.6	− 1.3	+28.1	+10.8
X 16	− 8.6	− 7.9	− 4.0	− 6.2	−22.6	− 8.6
X 17	− 5.7	− 5.2	− 1.5	+ 6.2	−17.1	− 5.7
Y 1	− 2.7	− 3.5	− 3.4	− 0.0	− 9.6	− 2.7
Y 3	− 5.4	− 6.8	− 6.9	− 0.0	−19.1	− 5.4
Y 5	− 8.1	−10.2	−10.3	− 0.0	−28.6	− 8.1
Y 7	−16.8	−21.2	− 3.3	− 9.4	−47.4	−16.8
Y 10	− 0.5	− 4.7	+ 8.6	+ 1.7	− 5.2	+ 8.1
Y 11	− 0.5	− 4.7	+ 8.6	+ 1.7	− 5.2	+ 8.1
Y 13	+ 0.5	− 4.7	+ 6.9	+ 3.4	− 4.2	+ 7.4
2 3	+ 1.1	+ 1.4	+ 1.4	+ 0.0	+ 3.9	+ 1.1
4 5	+ 2.2	+ 2.7	+ 2.8	+ 0.0	+ 7.7	+ 2.2
6 7	− 2.6	− 1.3	− 9.4	− 1.0	−13.3	− 2.6
8 9	− 2.2	− 2.7	− 2.8	− 0.0	− 7.7	− 2.2
10 11	− 0.0	− 0.0	− 0.0	− 0.0	− 0.0	− 0.0
12 13	− 0.0	− 0.0	+ 2.1	− 2.1	− 2.1	+ 2.1
14 14	− 0.0	− 0.0	− 0.0	− 0.0	− 0.0	− 0.0
17 17	+ 8.5	+ 5.3	+ 3.0	+ 3.4	+17.2	+ 8.5
1 2	− 2.9	− 3.7	− 3.8	− 0.0	−10.4	− 2.9
3 4	− 3.6	− 4.4	− 4.5	− 0.0	−12.5	− 3.6
5 6	− 4.3	− 5.4	− 5.5	− 0.0	−15.2	− 4.3
7 8	+ 0.6	− 1.9	+ 8.7	+ 1.4	− 1.3	+10.7
9 10	−14.4	−16.7	− 4.2	−10.2	−41.5	−14.4
11 12	+ 0.0	+ 0.0	− 2.7	+ 2.8	− 2.7	+ 2.8
13 14	+ 0.0	+ 0.0	− 2.8	+ 2.8	− 2.8	+ 2.8
15 16	+ 5.8	+ 5.4	+ 4.6	− 3.7	− 3.7	+15.8

Example 23.—Side Cantilever Trusses and Monitor

259. Description.—This roof covers a building with side aisles in two stories, the wide middle aisle being open to the roof; it would be suitable for a large shop or manufactory. A travelling crane might run on track beams attached to sides of middle columns next the middle aisle. Trusses over side aisles rest on outer walls and inner columns or walls, and they are extended as cantilevers to support a glazed monitor roof over the middle aisle for admission of light. Its roof is here assumed to be glazed as well as its side walls.



Figs. 198–201.—Cantilever and Skylight Truss.

260. Programme.—Compound truss of type as in Fig. 202; total span 200 ft.; divided into two two-story aisles each 50 ft. wide, and one central aisle 100 ft. wide; side slopes with rise of 35 ft. if continued to middle of building; monitor walls 15 ft. high; monitor roof 50 ft. span; rise 15 ft.; materials, steel; side roofs covered with felt and gravel on 1 3/8-inch white-pine sheathing; monitor walls and roof glazed on steel T-bars; no rafters; steel purlins and trusses;

trusses 25 ft. on centres; location at Milwaukee, latitude about 43° north; medium exposure.

261. Dimensions.—

$$\text{Tan } i = \frac{35}{100} = 0.3500 = \tan 19.3° \text{ for side roofs.}$$

$$\text{Tan } i = \frac{15}{25} = 0.6000 = \tan 31° \text{ for monitor roof.}$$

$$l = \frac{200}{16} = 12.5 \text{ ft. for side roofs.}$$

$$l = \frac{50}{6} = 8.34 \text{ ft. for monitor roof.}$$

$$l' = \frac{12.5}{\cos 19.3°} = 13.25 \text{ for side roofs.}$$

$$l'' = \frac{8.34}{\cos 31°} = 9.74 \text{ ft. for monitor roof.}$$

$A = 13.25 \times 25.00 = 331.3$ sq. ft. = apex area for side roofs.

$A = 15.00 \times 25.00 = 375.0$ sq. ft. = apex area for monitor wall.

$A = 9.74 \times 25.00 = 243.5$ sq. ft. = apex area for monitor roof.

262. Apex Loads.—Assume 100 ft. span for weight of side roofs; 50 ft. span for monitor.

$$\text{Truss} = \frac{100}{25} + \frac{100^2}{12600} = 4.794 \text{ lbs. per horizontal sq. ft. for side roof.}$$

$$\text{Truss} = \frac{50}{25} = \frac{50^2}{12600} = 2.198 \text{ lbs. per horizontal sq. ft. for monitor roof.}$$

Snow = 2.5 (43° − 35°) = 20.00 lbs. per horizontal sq. ft.

$$\text{Wind} = \frac{8}{9} \times 19.3° = 17.16 \text{ lbs. per inclined sq. ft. for side roofs.}$$

$$\text{Wind} = \frac{8}{9} \times 31° = 27.56 \text{ lbs. per inclined sq. ft. for monitor roofs.}$$

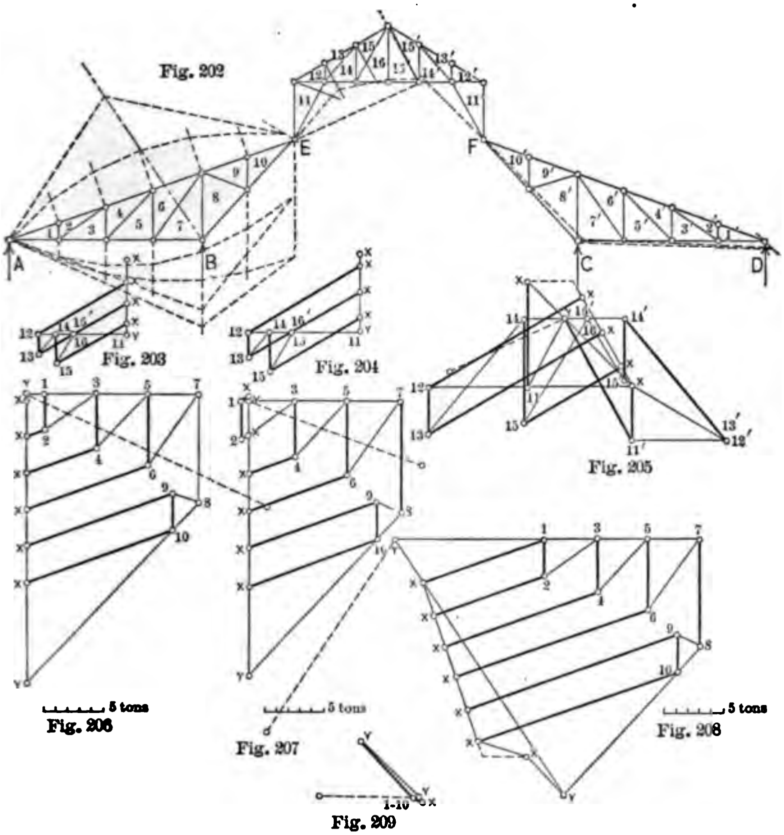
Wind = 40.00 lbs. per vertical sq. ft. for monitor walls.

$P = 331.3\ (6 + 4 + 0 + 4 + 4.794 \cos 19.3°) = 6139$ lbs. = 3.070 tons, side roof.

$P = 375.0\ (5 + 0 + 0 + 3 + 0) = 4500$ lbs. = 2.250 T., wall.

$P = 243.5\ (5 + 0 + 4 + 3 + 2.198 \cos 31°) = 3360$ lbs. = 1.680 T., monitor roof.

$S = 331.3\ (20.00 \cos 19.3°) = 6254$ lbs. $= 3.127$ T., side roof.
$S = 243.5\ (20.00 \cos 31°) = 4174$ lbs. $= 2.087$ T., monitor roof.
$W = 331.3 \times 17.16 = 5684$ lbs. $= 2.842$ T., side roof.
$W = 375.0 \times 40.00 = 15000$ lbs. $= 7.500$ T., wall.
$W = 243.5 \times 27.55 = 6708$ lbs. $= 3.354$ T., monitor roof.



FIGS. 202–209.—Cantilever and Monitor Trusses.

263. Total Loads on Half Truss.—

Permanent $= 3.069 \times 5\ 1/2 + 2.250 + 1.680 \times 3 = 24.17$ tons.
Snow $= 3.127 \times 5\ 1/2 + 2.087 \times 3 = 17.20$ tons.
Wind $= 2.842 \times 5\ 1/2 = 15.63$ tons for side roof.
Wind $= 7.500 \times 1 = 7.50$ tons for monitor wall.
Wind $= 3.354 \times 3 = 10.06$ tons for monitor roof.

264. Stress Diagrams.—It is here necessary to treat the monitor skylight separately, since the reactions at its ends compose a large portion of the loads at E and F supported by the side cantilever trusses. Figs. 203, 204, and 205 are the P, S, and W stress diagrams for the monitor alone.

After obtaining these reactions, they are laid off in the side cantilever truss $A\ B\ E$ as an additional load at E. The reactions for the truss $A\ B\ E$ are to be found by the equilibrium polygon for each stress diagram, as indicated in Fig. 202. Figs. 206, 207, and 208 are the P, S, and W stress diagrams for the truss $A\ B\ E$. Fig. 209 is the W stress diagram for the leeward cantilever truss $F\ C\ D$, when only supporting the leeward wind reaction of the monitor at F.

265. Stress Sheet for Example 23.—

Member.	*P*-stress.	*S*-stress.	*W*-stress *W*.	*W*-stress *L*.	Maximum.	Minimum.
X 1	+ 1.4	+ 0.4	+11.5	− 0.7	+13.3	+ 0.7
X 2	+ 1.4	+ 0.4	+10.6	− 0.7	+12.4	+ 0.7
X 4	+ 6.0	+ 4.2	+14.1	− 0.7	+24.3	+ 5.3
X 6	+10.5	+ 9.0	+17.6	− 0.7	+37.1	+ 9.8
X 9	+12.6	+10.0	+19.0	− 0.7	+41.8	+11.9
X 10	+12.6	+10.2	+18.0	− 0.7	+40.8	+11.9
X 11	− 5.0	− 6.3	− 9.0	+ 4.4	−20.3	− 0.6
X 12	− 8.0	−10.0	−14.3	+ 9.2	−32.5	+ 1.2
X 13	− 8.0	−10.0	−16.5	+ 9.2	−34.3	+ 1.2
X 15	− 6.4	− 8.0	− 9.0	+ 0.4	−23.4	− 6.0
Y 1	− 1.3	− 0.4	−14.0	+ 0.4	−15.7	− 0.9
Y 3	− 5.6	− 4.0	−18.1	+ 0.4	−27.7	− 5.2
Y 5	−10.0	− 8.4	−22.5	+ 0.4	−40.4	− 9.6
Y 7	−14.3	−13.0	−26.8	+ 0.4	−54.1	−13.9
Y 8	−20.6	−18.6	−17.2	− 7.5	−56.4	−20.6
Y 10	−17.5	−15.5	−14.5	− 7.5	−47.5	−17.5
Y 11	− 0.0	− 0.0	+ 6.6	−11.5	−11.5	+ 6.6
Y 14	+ 5.5	+ 7.0	+ 3.0	− 5.3	+15.5	+ 0.2
Y 16	+ 4.0	+ 5.1	+ 2.4	− 2.2	+ 9.1	+ 1.8
1 2	− 3.0	− 3.0	− 3.0	− 0.0	− 9.0	− 3.0
3 4	− 4.5	− 4.7	− 4.5	− 0.0	−13.7	− 4.5
5 6	− 6.0	− 6.3	− 6.0	− 0.0	−18.3	− 6.0
7 8	− 9.3	− 9.5	− 9.3	− 0.0	−28.1	− 9.3
9 10	− 2.2	− 3.1	− 3.0	− 0.0	− 8.3	− 2.2
12 13	− 1.7	− 2.0	− 3.9	− 0.0	− 7.6	− 1.7
14 15	− 2.5	− 3.0	− 8.8	+ 0.5	−14.3	− 2.0
16 16′	− 0.0	− 0.0	− 0.0	− 0.0	− 0.0	− 0.0
2 3	+ 5.2	+ 5.4	+ 5.2	+ 0.0	+15.8	+ 5.2

Member.	*P*-stress.	*S*-stress.	*W*-stress *W*.	*W*-stress *L*.	Maximum.	Minimum.
4 5	+ 6.2	+ 6.5	+ 6.3	+ 0.0	+19.0	+ 6.2
6 7	+ 7.5	+ 7.7	+ 7.4	+ 0.0	+22.6	+ 7.5
8 9	+ 2.4	+ 2.4	+ 2.4	+ 0.0	+ 7.2	+ 2.4
11 12	+ 7.0	+ 8.5	+ 7.8	− 7.7	+23.3	− 0.7
13 14	+ 2.3	+ 2.7	+12.7	−13.2	+17.7	−10.9
15 16	+ 3.0	+ 3.5	+10.3	+ 6.4	+16.8	+ 3.0

Example 24.—Octagonal Hip Roof with Lantern

266. Description.—This roof has the form of an octagonal pyramid over an octagonal room, and it has an octagonal lantern at top with a double window in each side and a hip roof. The roof and its loads are supported by rafters and purlins, attached to the hip principals, on which rests nearly the entire weight of the roof. There are no trusses, so that the stresses in the different members are obtained graphically, while their dimensioning is deferred to a later chapter, after the introduction of the necessary formulas and tables.

267. Programme.—Internal diameter of octagonal roof 80 ft.; external diameter 84 ft.; internal diameter of lantern 15 ft., with external diameter of 17 ft.; vertical base of roof 5 ft. high; total height of pyramid to its apex 42 ft., with a visible height of 33.3 ft.; exterior of lantern 6 ft. high; roof of lantern 8.6 ft. high; all roofs are inclined at 45°; material, steel; covering of slate on felt (12 #) on 3-inch slab of reënforced concrete (37.5 #); inside ceiling of plaster on steel bars suspended from roof (12 #); main hip principals at angles to support framework; horizontal purlins 9.3 ft. on centres; rafters 4 ft. on centres; double window (16 sq. ft.) in each side of lantern; ceiling of lantern horizontal; location at Detroit, Mich., latitude about 42 1/2° north; maximum exposure.

268. Dimensions.—

Tan i = tan 45° by construction.

$$l = \frac{40.0-8.5}{5} = 6.50 \text{ ft.} = \text{horizontal projection of panel of}$$

rafter of main roof.

$$l' = \frac{6.5}{\cos 45^\circ} = 9.19 = \text{length of panel of rafter of same.}$$

$l'' = \dfrac{8.5}{\cos 45°} = 12.02$ ft. = slant height of lantern roof.

$A = 5.0 \times 4.0 = 20.00$ sq. ft. = apex area of vertical base.

$A = 9.19 \times 4.0 = 36.76$ sq. ft. = apex area on rafter.

$A = 7.0 \times 6.0 = 42.00$ sq. ft. = area of side of lantern wall.

$A = \dfrac{7.00 \times 12.02}{2} = 42.07$ sq. ft. = area of side of lantern roof.

269. Apex Loads.—

Framework $= \dfrac{80}{25} + \dfrac{80^2}{12600} = 3.71$ lbs. per horizontal sq. ft.

Snow $= 2.5\ (42\ 1/2° - 35°) = 18.75$ lbs. per horizontal sq. ft.

Wind = 50 lbs. per sq. ft. of vertical and inclined surfaces.

$P = 20.00\ (12 + 37.5 + 4 + 3 + 12 + 3.71 \cos 45°) = 1422$ lbs. = 0.711 tons for base.

$P = 35.76\ (12 + 37.5 + 4 + 3 + 12 + 3.71 \cos 45°) = 2543$ lbs. = 1.277 tons main roof.

$P = 26.00\ (12 + 37.5 + 4 + 3 + 12 + 3.71 \cos 45°) = 1849$ lbs. = 0.925 tons lantern wall.

$P = 42.07\ (12 + 37.5 + 4 + 3 + 12 + 3.71 \cos 45°) = 2992$ lbs. = 1.496 tons lantern roof.

$S = 36.76\ (18.75 \cos 45°) = 487$ lbs. = 0.244 tons for main roof.

$S = 42.07\ (18.75 \cos 45°) = 558$ lbs. = 0.279 tons for lantern roof.

$W = 20.00 \times 50.00 = 1000$ lbs. = 0.500 tons for vertical base.

$W = 35.76 \times 50.00 = 1788$ lbs. = 0.894 tons for main roof.

$W = 42.00 \times 50.00 = 2100$ lbs. = 1.050 tons for side of lantern wall.

$W = 42.07 \times 50.00 = 2104$ lbs. = 1.052 tons for side of lantern roof.

270. Stress Diagrams.—These are employed to determine the maximum transverse bending moment and longitudinal compression acting on each member of the framework, in order later to compute by formulas the proper dimensions of each.

Rafter of lantern roof supports half area and loads on one side of that roof. Hence its loads are: $P = \dfrac{1.496}{2} = 0.748$; $S = \dfrac{0.279}{2} =$

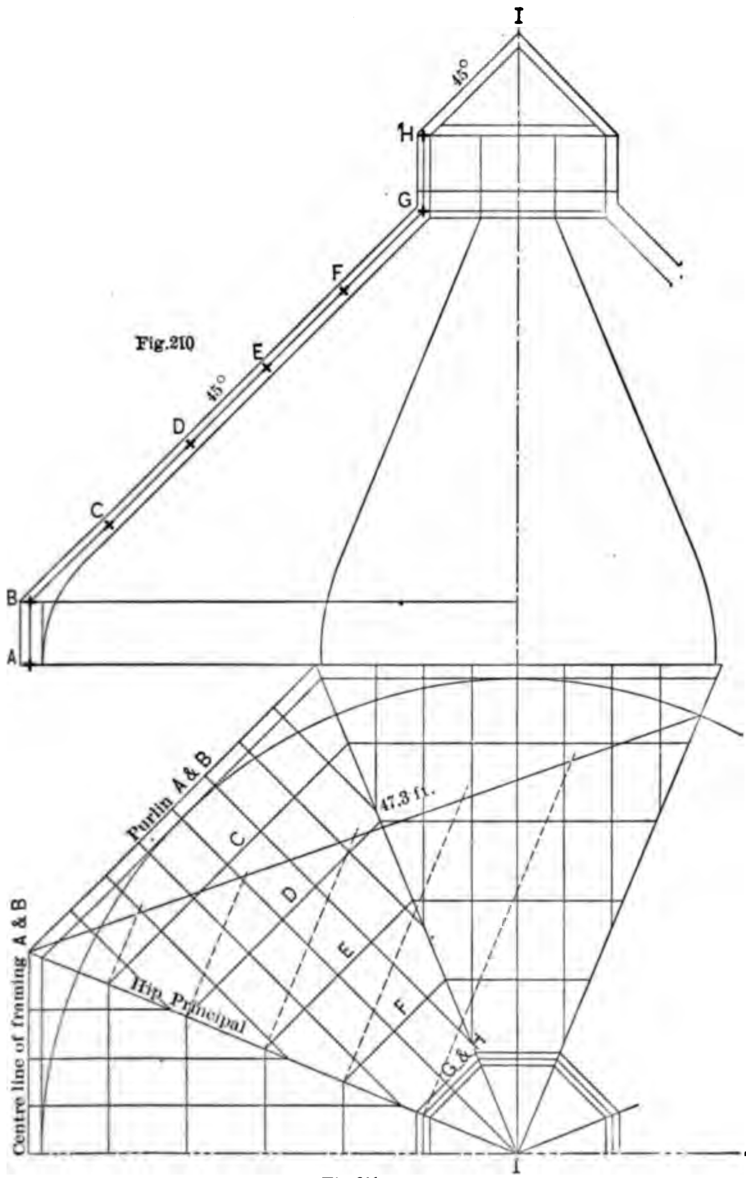


Fig.211 .

Fig. 211.—Octagonal Hip Roof with Lantern.

0.140; $W = \frac{1.052}{2} = 0.526$; centre of gravity of each load is at $\frac{12.02}{3} = 4.01$ ft. from lower end of rafter.

By Fig. 212, the total normal load (perpendicular to) on rafter = 1.18 T., the total parallel (lengthwise) load on rafter = 0.45 T. producing longitudinal compression.

By Fig. 213, $M_{max} = 5.00 \times 0.5 = 2.5$ ft.-tons for a concentrated load. The actual value of M_{max} is somewhat less, since the load uniformly diminishes from the lower end to 0 at the upper end.

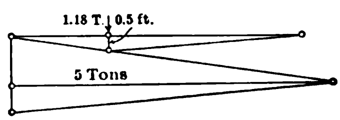


FIG. 213

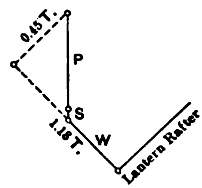


Fig. 212

Hip rafter of lantern roof supports the same area, so that its loads are the same, as well as its transverse and longitudinal stresses.

Rafter of main roof supports $P = 1.277$ T., $S = 0.237$ T., $W = 0.894$ T.

By Fig. 214, normal load on rafter = 1.96 T. and parallel load = 1.04 T., both components being uniformly distributed along its length.

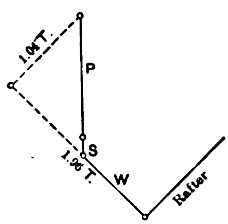


Fig. 214

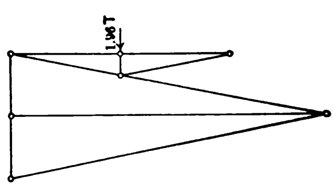


Fig. 215

By Fig. 215, $M_{max} = \frac{5.00 \times 0.95}{2} = 2.38$ ft.-tons for uniform load.

Purlin H at base of side of lantern roof supports rafter at middle. Length = 7.00 ft.; load same as load on rafter.

By Fig. 216, vertical component of load at middle = 1.25 T., horizontal component = 0.40 T.

By Fig. 217, vertical $M_{max} = 5.00 \times 0.5 = 2.50$ ft.-tons; horizontal $M_{max} = \frac{2.50 \times 0.40}{1.25} = 0.80$ ft.-tons, and adding 0.46 for moment of wind on half lantern wall below, total $M_{max} = 1.26$ ft.-tons.

Purlin G at base of lantern wall. Deduct for window for P load, and taking the load as uniformly distributed, $P = 0.925$ T., $W = 1.05$ T. Add half the load on one rafter at middle $= P = \frac{1.277}{2} = 0.639$ T. $S = \frac{0.237}{2} = 0.119$ T. $W = \frac{0.894}{2} = 0.447$ T.

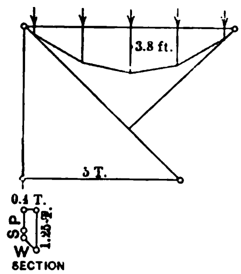


Fig. 216

By Fig. 218, vertical component of load = 2.00 T., horizontal component = 1.42 T.

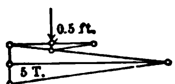


Fig. 217

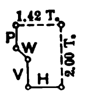


Fig. 218

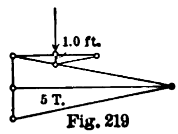


Fig. 219

By Fig. 219, vertical $M_{max} = 5.00 \times 1.00 = 5.00$ ft.-tons; horizontal $M_{max} = \frac{5.00 \times 1.42}{2.00} = 3.50$ ft.-tons.

Purlin F supports 3 rafters and their loads. Length = 12.25 ft. It will be more convenient for framing these purlins to set them with webs perpendicular to slope of roof, then obtaining the normal and parallel values for M_{max}.

By Fig. 214, $1.96 \times 3 = 5.88$ T. = normal load on purlin.
$1.04 \times 3 = 3.12$ T. = parallel load on purlin.

By Fig. 220, normal $M_{max} = 5.00 \times 2.00 = 10.00$ ft.-tons.

Then parallel $M_{max} = \frac{10.00 \times 3.12}{5.88} = 5.30$ ft.-tons.

Purlin E supports 5 rafters and their loads. Length = 17.65 ft.

By Fig. 214, $1.96 \times 5 = 9.80$ T. = normal load on purlin.
$1.04 \times 5 = 5.20$ T. = parallel load on purlin.

By Fig. 221, normal $M_{max} = 9.80 \times 3.8 = 37.20$ ft.-tons.

Then parallel $M_{max} = \frac{37.20 \times 5.20}{9.80} = 20.20$ ft.-tons.

Purlin D supports 5 rafters and their loads. Length = 23.05 ft. Then, as for purlin E, 9.80 T. = normal component and 5.20 T. = parallel component.

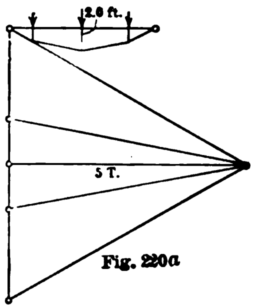


Fig. 220a

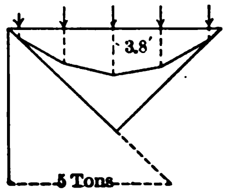


FIG. 221

By Fig. 222, normal $M_{max} = 9.80 \times 6.50 = 63.65$ ft.-tons.

Then parallel $M_{max} = \dfrac{63.65 \times 5\ 20}{9.80} = 33.77$ ft.-tons.

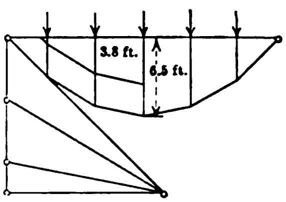


Fig. 222

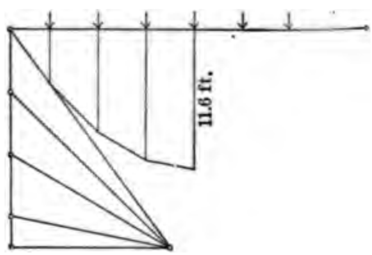


Fig. 223

Purlin C supports 7 purlins and their loads. Length = 28.5 ft.
By Fig. 214, $1.96 \times 7 = 13.71$ tons = normal component.
By Fig. 214, $1.04 \times 7 = 7.28$ tons = parallel component.
By Fig. 223, normal $M_{max} = 5.00 \times 11.6 = 58.00$ ft.-tons.

Then parallel $M_{max} \dfrac{58.00 \times 7.28}{13.71} = 30.08$ ft.-tons.

Purlin B supports 7 purlins and loads, together with half the W loads on the vertical base. It is best to make the axis of section vertical.

Length = 33.0 ft.

Then $\dfrac{13.71}{2} = 6.86$ tons = normal component.

And $\dfrac{7.28}{2} = 3.64$ tons = parallel component.

Also $\dfrac{5 \times 28.5 \times 50}{2 \times 2000} = 1.78$ tons = horizontal W load on half the base.

By Fig. 224, total vertical component = 7.40 tons.

By Fig. 224, total horizontal component = 1.78 tons.

By Fig. 225, vertical $M_{max} = 7.40 \times 7.2 = 53.28$ ft.-tons.

Also, horizontal $M_{max} = 1.78 \times 7.2 = 12.81$ ft.-tons.

If this purlin is supported by verticals beneath the end of each rafter, the vertical M_{max} would not occur and might be neglected. The horizontal M_{max} would still remain to require safe horizontal resistance in the purlin.

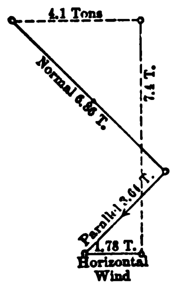


FIG. 224

Purlin A. Since this would rest directly on the enclosing walls and is bolted to them, no calculations are required for it.

Hip rafters of lantern roof. Supports same area and load as the slant rafter, but its length would be increased to 12.7 ft.

Angle vertical of lantern supports a vertical load of 2.00 tons.

Hip rafter of main roof is 47.3 ft. long, being divided into 5 equal panels of 9.46 ft. each. Inclination is 43°. It supports the ends of

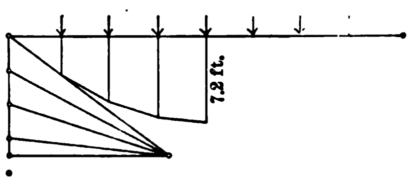


Fig. 225

4 pairs of purlins. The horizontal component or reaction at upper end produces compression in the ring purlins at G, this ring being finally supported by longitudinal compression in the 8-hip rafters.

271. Stress Sheet for Example 24.—

Lantern hip rafter.	$P = 1.496$;	$S = 0.279$;	$W = 1.042$ tons.
Lantern wall.	$P = 0.925$;	$S = 0.000$;	$W = 1.050$ tons.
For purlin G.	$P = 2.421$;	$S = 0.297$;	$W = 2.102$ tons.
For purlin F.	$P = 5.700$;	$S = 1.050$;	$W = 4.050$ tons.
For purlin E.	$P = 9.500$;	$S = 1.750$;	$W = 6.750$ tons.
For purlin D.	$P = 9.500$;	$S = 1.750$;	$W = 6.750$ tons.
For purlin C.	$P = 13.500$;	$S = 2.450$;	$W = 9.450$ tons.

Purlins A and B are supported otherwise.

By Fig. 227, at G; normal component = 4.06 tons.

By Fig. 227, at G; parallel component = 1.90 tons.

Or vertical component = 4.30 tons.

And horizontal component = 1.48 tons.

By Fig. 228, at F; normal component = 9.00 tons.

By Fig. 228, at F; parallel component = 4.55 tons.

By Fig. 229, at E; normal component = 14.90 tons.

By Fig. 229, at E; parallel component = 7.55 tons.

By Fig. 229, at D, adding one purlin load for roof directly support-

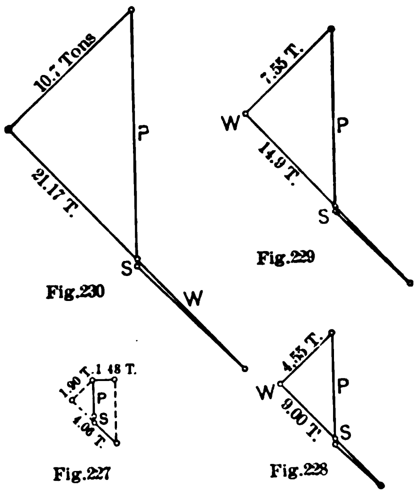


Fig. 230 Fig. 229 Fig. 227 Fig. 228

ed; normal component = 17.38 tons; parallel component = 8.81 tons.

By Fig. 230, at C; normal component = 21.10 tons.

By Fig. 230, at C; parallel component = 10.70 tons.

By Fig. 231, normal M_{max} for hip rafter = 40.00 × 11.6 = 464.00 ft.-tons.

CHAPTER V

TYPICAL ROOF TRUSSES AND STRESS DIAGRAMS

272. Purpose of these Examples.—From the latest technical works in English, French, and German have been selected about 70 typical forms of roof trusses, comprising those likely to occur in practice. A few of these have been fully studied as examples in Chapter IV. In order to exhibit the form of the corresponding stress diagram and to suggest the proper mode of solving any difficulties in treating them, unit apex permanent loads have been assumed and the permanent stress diagram worked out for the left-hand half of each truss. The snow and wind stress diagrams may then be easily completed by proceeding in a similar manner. Since these stresses would only be measured in units and their fractions, all stress sheets are necessarily omitted.

273. Examples.—Fig. 232 is a very common type, which has been considered in Example 1 of Chapter IV.

Fig. 234 is similar, but has a cambered lower chord, which evidently increases the stresses in the chords and middle vertical, as seen by comparing Figs. 233 and 234.

Fig. 236 has reversed diagonals, which fact reverses the nature of the stresses in web members and changes the shape of the stress diagram. (Fig. 237.)

Fig. 238 is similar to the last, but with cambered lower chord, increasing stresses in chords and middle vertical.

Fig. 240 is a peculiar type with raised lower chord and trussed principals.

Fig. 242 has a depressed lower chord, making the stresses in lower chord slightly smaller, but its bad appearance would be objectionable.

In Fig. 244, the apexes of the lower chord are horizontally midway between those of the upper chord. It might be termed the Howe type.

Fig. 246 is similar, but with cambered lower chord.

Fig. 248 is a common form of Fink truss, that has been worked out in Example 3 of Chapter IV.

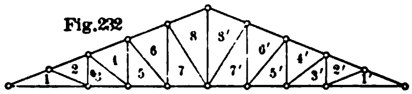


Fig. 232

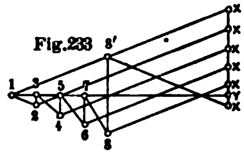


Fig. 233

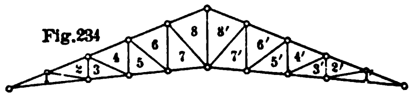


Fig. 234

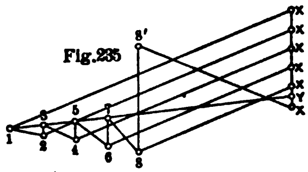


Fig. 235

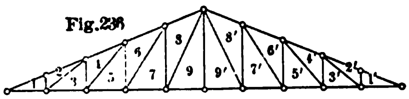


Fig. 236

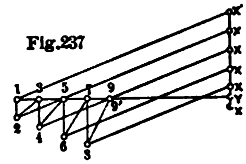


Fig. 237

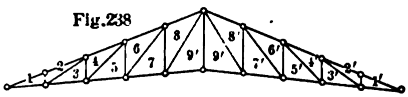


Fig. 238

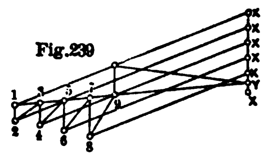


Fig. 239

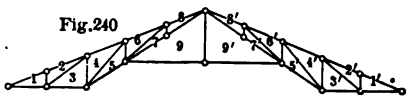


Fig. 240

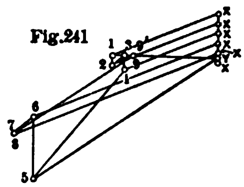


Fig. 241

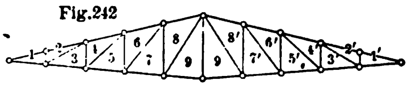


Fig. 242

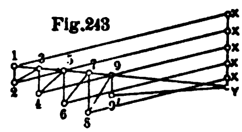


Fig. 243

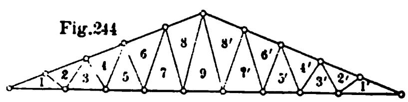


Fig. 244

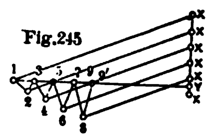


Fig. 245

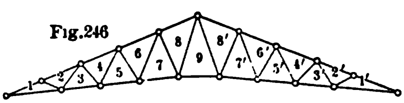


Fig. 246

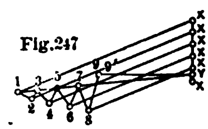


Fig. 247

Fig. 250 has a cambered lower chord, increasing the stresses. (Fig. 251.)

Fig. 252 is a Fink truss with raised lower chord.

Fig. 254 is a Fink truss with 16 panels for wider spans and with raised lower chord. A similar truss with cambered lower chord was treated in Example 5 of Chapter IV.

Fig. 256 is an unsymmetrical Fink truss of 6 panels for roof with equal inclinations and resting on walls of unequal heights.

Fig. 258 is a similar truss with 12 panels, which was fully worked out in Example 7 of Chapter IV. Such a form of truss might be required for an ore-mill with floors at different levels, in order to pass the materials downward by gravity.

Fig. 260 represents a modified Fink truss with 10 panels, for use when one with 8 or 16 panels might be inconvenient. See Example 6, Chapter IV.

Fig. 262 is the same type with cambered lower chord.

Fig. 264 is similar, but with raised lower chord.

Fig. 266 is a type of truss employed in Germany for the same purpose as the common saw-tooth roof over shops, etc., but possessing many advantages over the latter. The steeper slopes at the middle are glazed on one or both sides, admitting an abundance of light. External rain and snow drain onto the lower slopes, and internal drip gutters may easily be arranged to discharge thereon, thus avoiding the danger of leakage and the necessity of shoveling off the snow from the roof.

Fig. 268 is similar, with raised lower chord for sake of better appearance.

Fig. 270 represents one bay of the ordinary saw-tooth roof truss.

Fig. 272 is a German type of roof truss for a church; it has a cambered lower chord, which would appear best if made of steel.

Fig. 274 is a scissors truss for the same purpose. The stress in the middle vertical is here very large. A horizontal diagonal is sometimes substituted therefor, but has an inferior appearance.

Fig. 276 is a combined scissors and Fink truss for same purpose.

Fig. 278 is a scissors truss with cambered and arched lower chord for a church roof, which might be of steel and also support an arched ceiling attached to the lower chord.

Fig. 280 is a peculiar mansard type with raised lower chord, the roof having tiled or slated sides and deck covered with tin or felt and gravel.

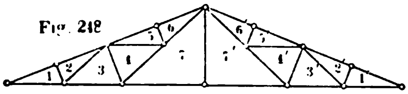


Fig. 248

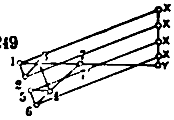


Fig. 249

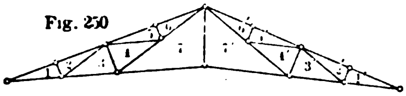


Fig. 250

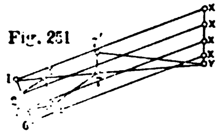


Fig. 251

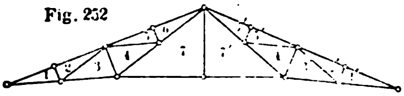


Fig. 252

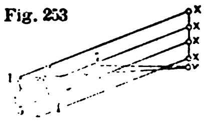


Fig. 253

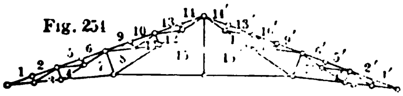


Fig. 254

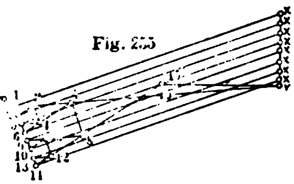


Fig. 255

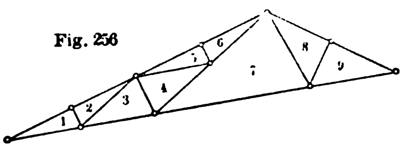


Fig. 256

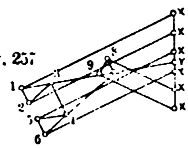


Fig. 257

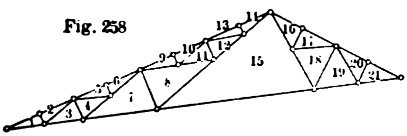


Fig. 258

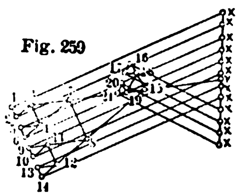


Fig. 259

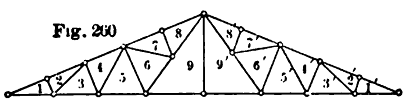


Fig. 260

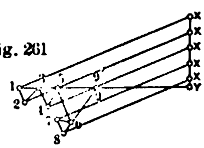


Fig. 261

Fig. 262

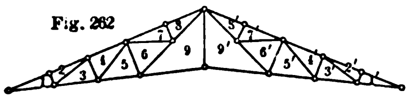


Fig. 263

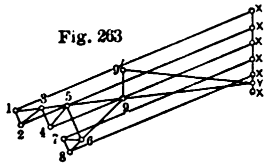


Fig. 264

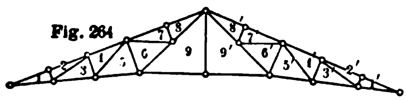
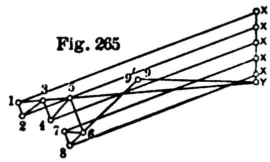


Fig. 265



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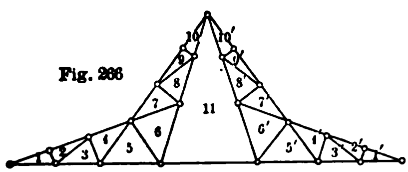

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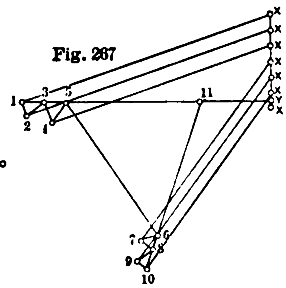

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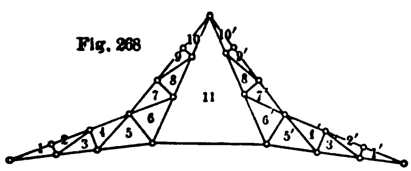

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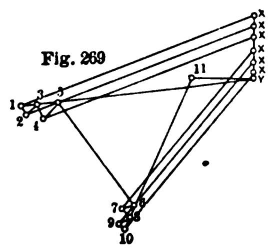
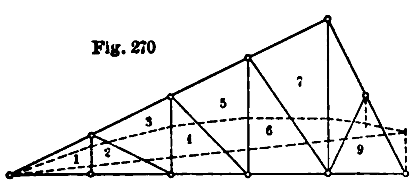


Fig. 270



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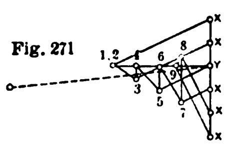

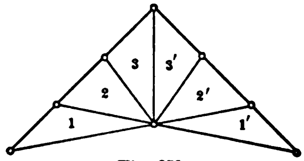

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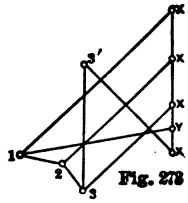

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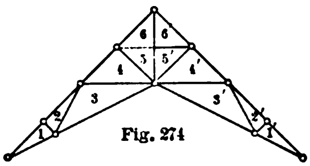


Fig. 274

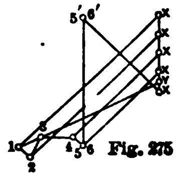


Fig. 275

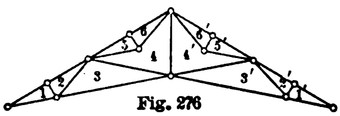


Fig. 276

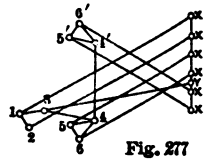


Fig. 277

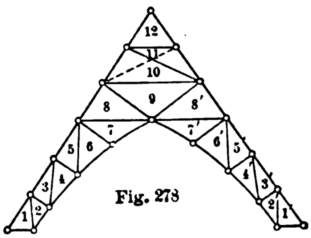
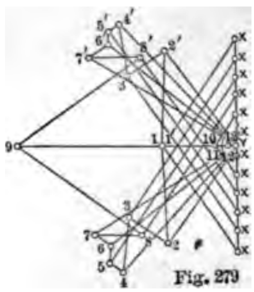


Fig. 278



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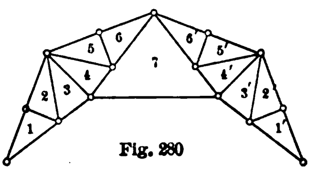


Fig. 280

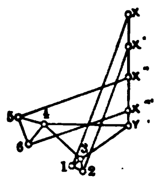


Fig. 281

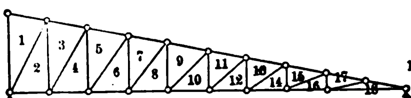


Fig. 282

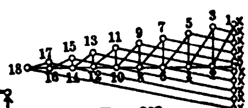


Fig. 283

Fig. 282 supports a shed roof with a single slope.

Fig. 284 is similar, with reversed diagonals.

Fig. 286 has reversed diagonals on right half only.

Fig. 288 is a shed-roof truss with curved lower chord.

Fig. 290 is a truss composed of two trussed principals and a tie at middle.

Fig. 292 supports slated sides with tinned deck and a central glazed roof for lighting the interior of the buildings.

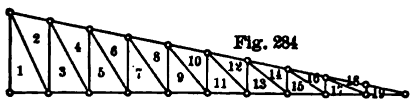


Fig. 284

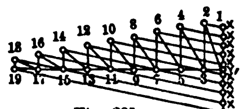


Fig. 285

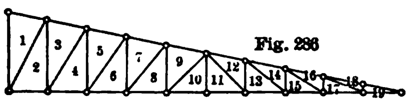


Fig. 286

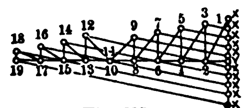
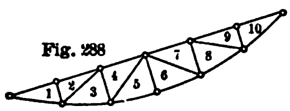


Fig. 287



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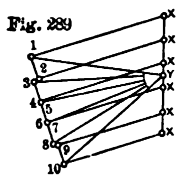

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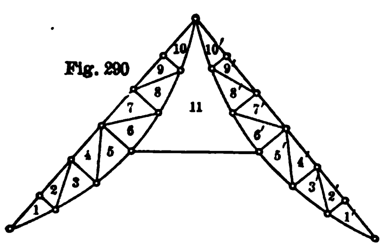


Fig. 290

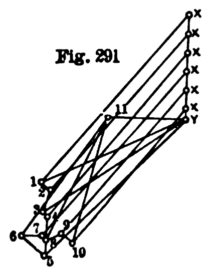


Fig. 291

Fig. 294 has a curved lower chord with diagonals inclined inwards.

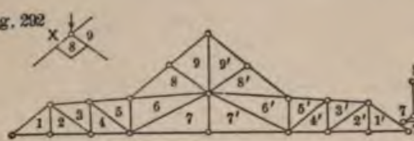
Fig. 296 has diagonals inclined outwards.

Fig. 298 has its upper chord in form of a circular arc with straight lower chord. The stresses in members of lower chord are nearly uniform and those in web members are very small.

Fig. 300 is similar to the last, but with reversed diagonals.

In the two last trusses, if the curve of the upper chord be a parabola with vertex at middle of span, and if the apex loads are equal, no stress occurs in web members. Hence in this form of truss

Fig. 292



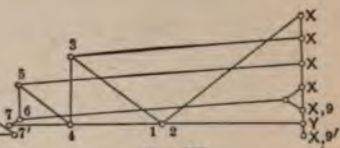


Fig. 293

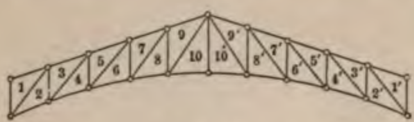


Fig. 294

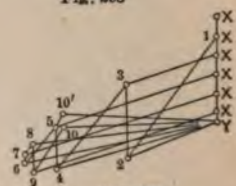


Fig. 295

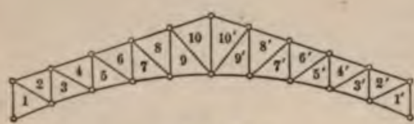


Fig. 296

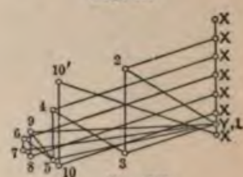


Fig. 297

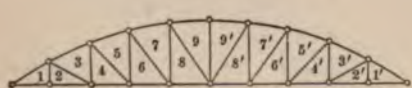


Fig. 298

Fig. 299

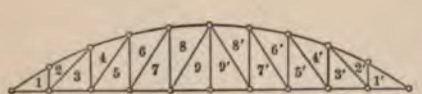


Fig. 300

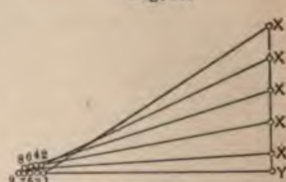


Fig. 301

the web members merely resist stresses produced by unequal apex loads (wind) or by accidental irregular loading.

Fig. 302 is a type of crescent truss with glazed monitor over the middle panels, suitable for a railway train-shed or shops. A simple crescent truss is fully treated in Example 9 of Chapter IV.

Fig. 302

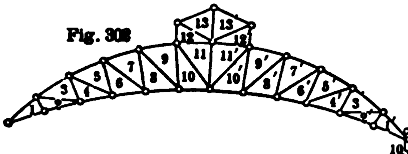


Fig. 303

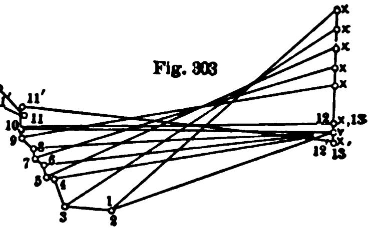


Fig. 304

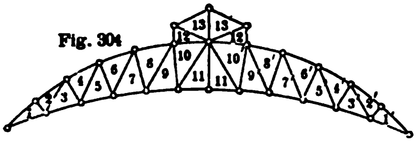


Fig. 305

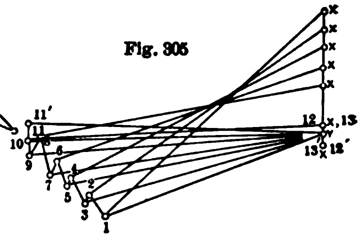


Fig. 306

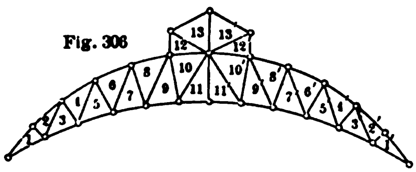


Fig. 307

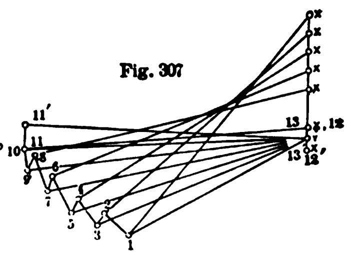


Fig. 308

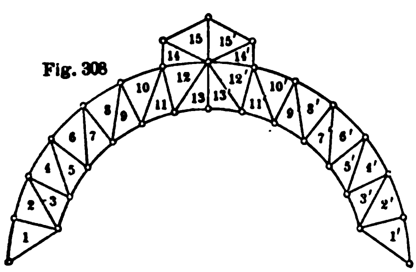
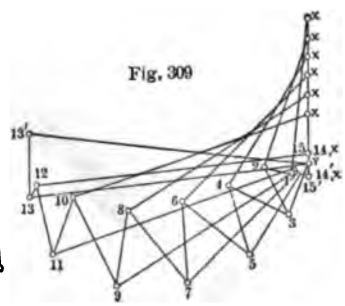


Fig. 309



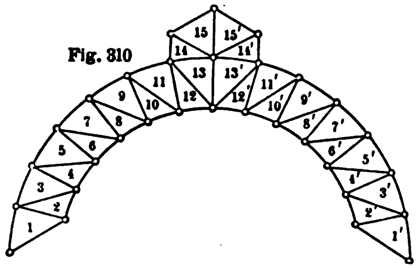


Fig. 310

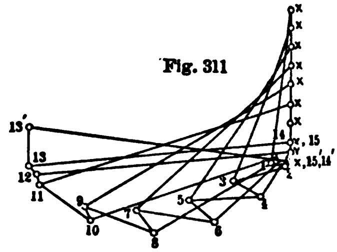


Fig. 311

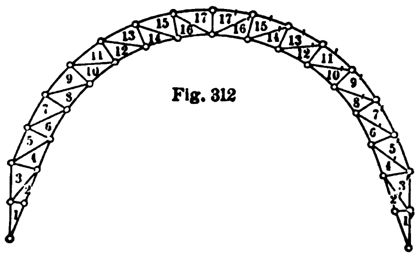


Fig. 312

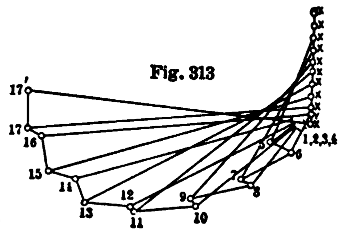


Fig. 313

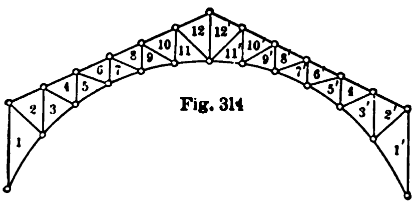


Fig. 314

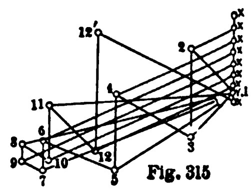


Fig. 315

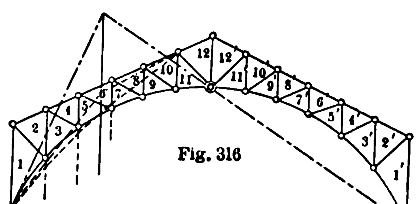


Fig. 316

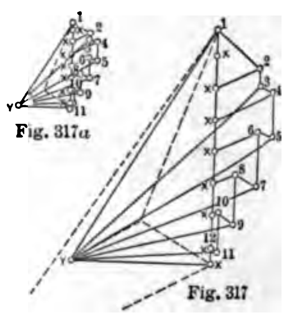


Fig. 317a

Fig. 317

Fig. 304 is similar, but with reversed diagonals.

Fig. 306 has somewhat different proportions and stresses.

Fig. 308 is a semicircular truss with parallel chords and a monitor.

Fig. 310 is similar, but with reversed diagonals.

Fig. 312 is nearly semicircular and with parallel chords, excepting for the two lower panels on each side. Its ends rest on two pin joints. It would be an excellent type for a train-shed, though the stresses in chords would be rather large.

Fig. 314 has vertical ends, the upper chord having uniform inclinations, while the lower chord is a circular arc. Verticals are equidistant horizontally. Pin joints at ends only, and not at top.

Fig. 316 is of similar form, but has three pin joints, one being at middle apex of lower chord. The middle apex of the upper chord does not connect the two adjacent X-members, but it is split, their ends being supported by separate struts 12 12′ to the pin joint. Trusses with 3 pin joints are treated in Examples 15 and 16 of Chapter IV. Such trusses are more economical than similar types with 2 pin joints at ends (Fig. 306) if stable supports are provided safely to resist considerable horizontal thrust at each end.

Fig. 318 has the pin joint at middle apex of upper chord, the middle apex of lower chord being split.

Fig. 320 is a truss with three hinges, composed of two trussed principals with 3 pin joints, the vertical walls being enclosed. An economical type for wide spans.

Fig. 322 is of similar character with reversed diagonals.

Fig. 324 is a crescent truss with 3 pin joints, the middle apex of the upper chord being split and disconnected.

Fig. 326 is a type similar to the double lenticular bridge truss, composed of a primary truss, Fig. 328, and two trussed principals, Fig. 327, the upper chord forming a circular arc with 3 joint pins. The lower chord is merely suspended by extending the verticals. Fig. 327*a* is the stress diagram for the left principal, Fig. 327.

Fig. 330 is a truss with cambered lower chord, supported at apexes *A* and *B*, thus having cantilever ends. It is evidently a very economical type, as shown by the half-stress diagram, Fig. 325.

Fig. 332 is similar, with reversed diagonals.

Fig. 334 also has cantilever ends and a raised middle lower chord. Also quite economical.

Fig. 336 is similar, but is intended for wider spans.

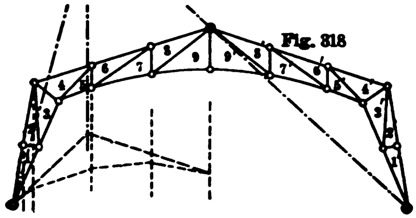


Fig. 318

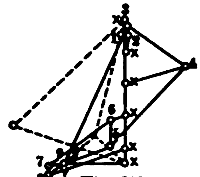


Fig. 319

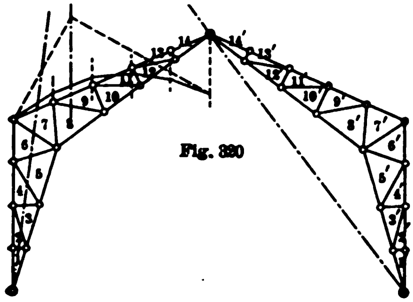


Fig. 320

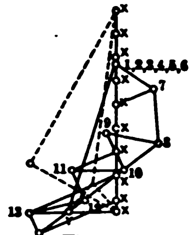


Fig. 321

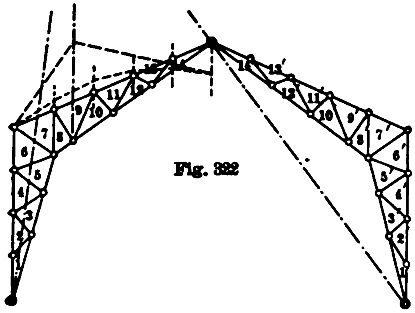


Fig. 322

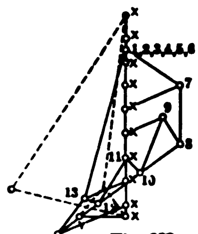


Fig. 323

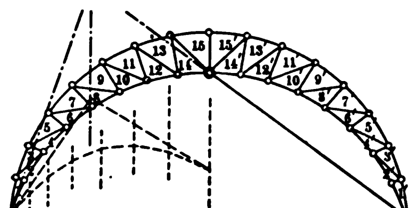


Fig. 324

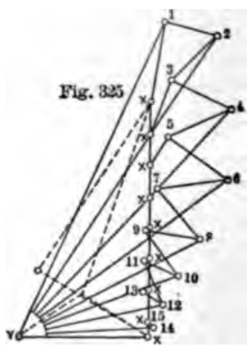


Fig. 325

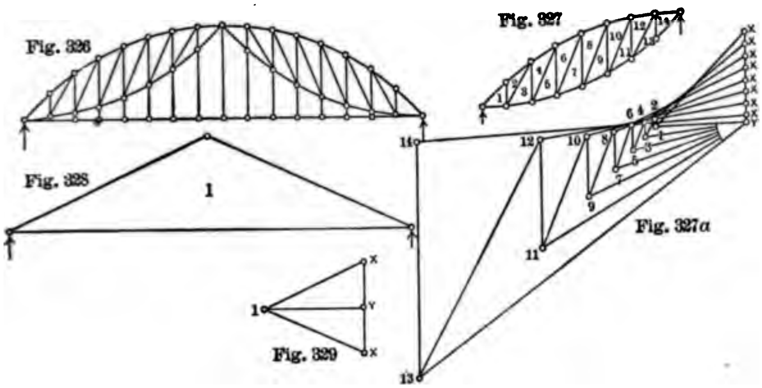


Fig. 326

Fig. 327

Fig. 327a

Fig. 328

Fig. 329

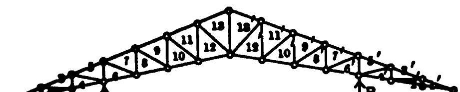


Fig. 330

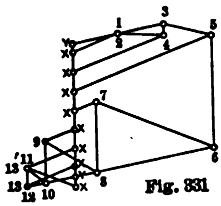
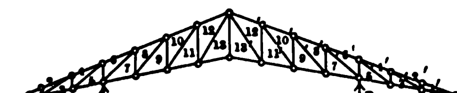


Fig. 331



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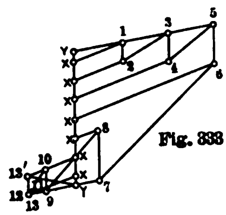

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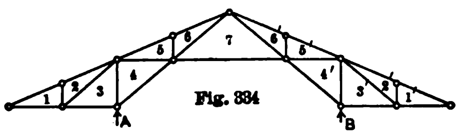

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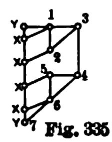

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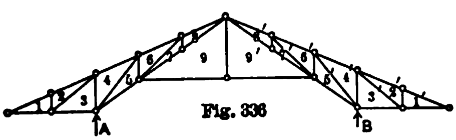

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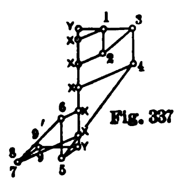

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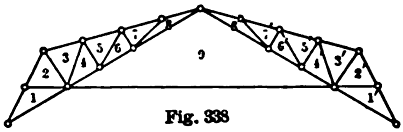

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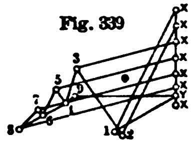

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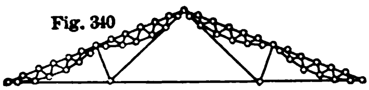

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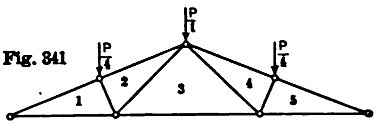

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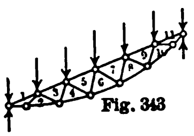

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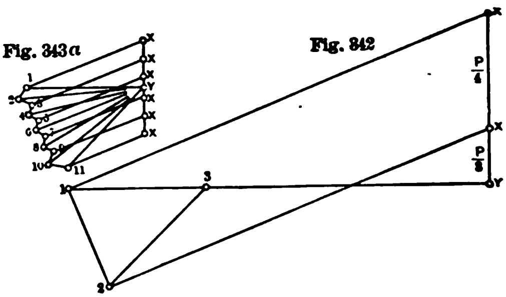
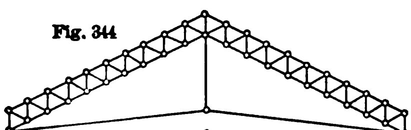


Fig. 343a

Fig. 342



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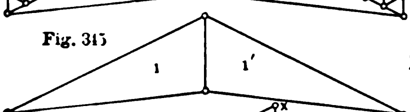


Fig. 345

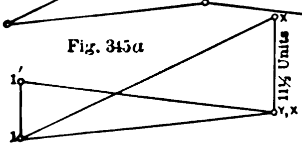


Fig. 345a

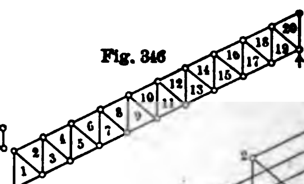


Fig. 346

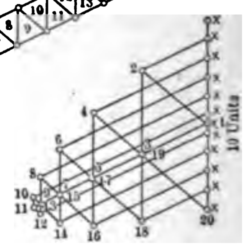


Fig. 347

Fig. 338 is a mansard truss, consisting of trussed principals and raised lower chord.

Fig. 340 is a compound truss, consisting of a 4-panel Fink truss with trussed members of upper chord. Figs. 341 and 342 are the truss and half-stress diagrams for the primary truss; Fig. 343*a* is the stress diagram for the trussed member in Fig. 343. This type was fully studied in Example 18 of Chapter IV.

Fig. 344 is also a compound truss, composed of a primary truss, Fig. 345, with trussed principals, Fig. 346. Fig. 345*a* is the half-stress diagram for the primary truss, and Fig. 347 is the stress diagram for the latticed principal in Fig. 346.

Fig. 348 is a truss with cantilever ends and central glazed roof.

Fig. 350 consists of a simple truss and two cantilevers fixed to the columns or walls supporting the ends of the middle truss.

Fig. 352 is a truss with cantilever ends.

Fig. 354 is similar, but with parallel chords.

Fig. 356 has a circular lower chord.

Fig. 358 is like the last, but with reversed diagonals.

Fig. 360 supports a shed roof with projecting cantilever, which slopes back to valley over the columns. This might cover the portico of a railway station, the cantilever overhanging the track.

Fig. 362 is a type with two cantilevers and supported by two columns, having a middle valley for rain water. Suitable for middle platform between two tracks.

Fig. 364 is for a similar purpose, but covering a much wider platform between two tracks, or it might be employed for a train-shed between two different railways.

Fig. 366 is intended for a wide train-shed supported by rows of columns, and it may be repeated as often as necessary. The truss has pin joints at C, A, and B, though for train-sheds over 100 ft. wide, expansion joints would be required at A or B.

Fig. 368 is the truss $C\,D\,A$ enlarged, Fig. 367 being the corresponding stress diagram, Figs. 370 and 369 are the truss and half-stress diagrams for the simple truss $A\,B$.

Fig. 372 comprises a simple truss with its stress diagram in Fig. 373, while Fig. 374 is the stress diagram for the lower portion $C\,B$ resting on two supports.

Fig. 379 represents a central glazed monitor supported by the cantilever ends of the side trusses. Figs. 380 and 383 are the truss and stress diagrams for the side trusses, Figs. 381 and 382 being those

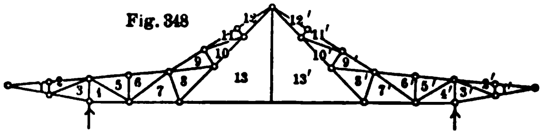
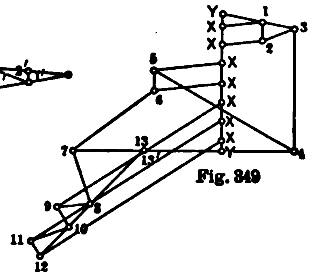


Fig. 348



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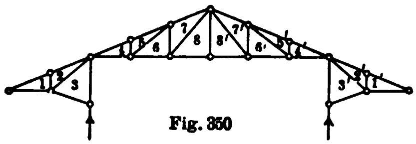
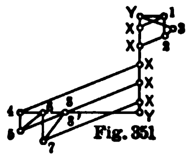


Fig. 350



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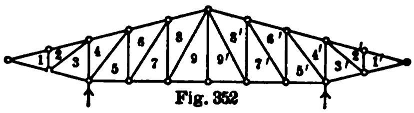


Fig. 352

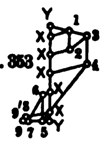


Fig. 353

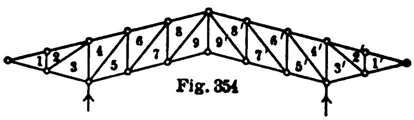


Fig. 354

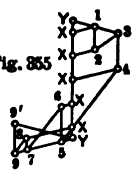


Fig. 355

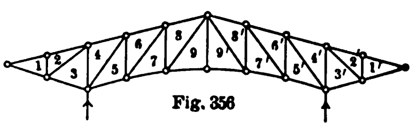
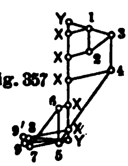


Fig. 356



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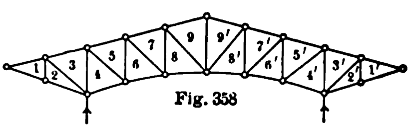

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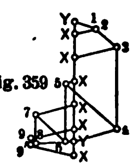

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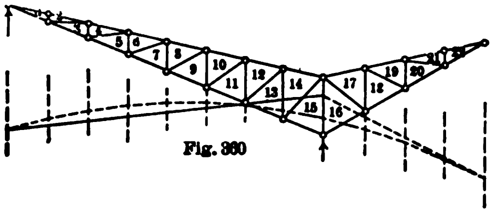
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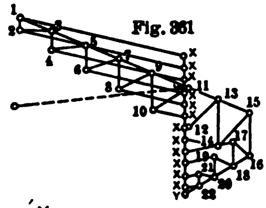
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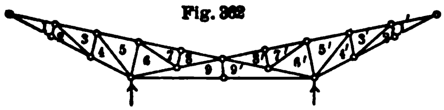
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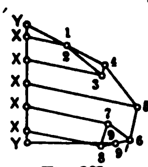
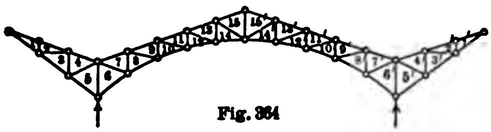


Fig. 363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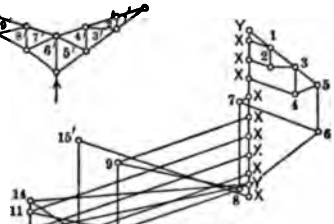
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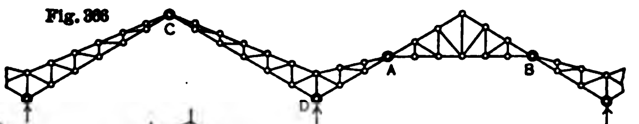

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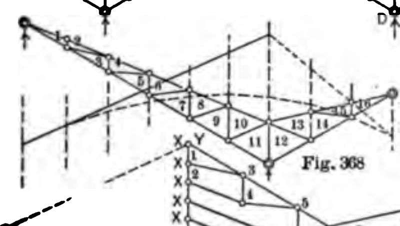
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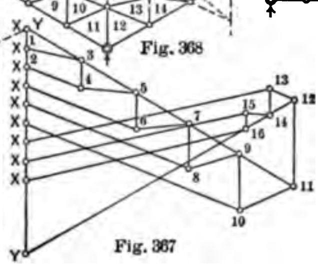
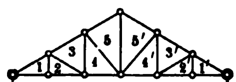


Fig. 367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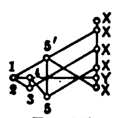
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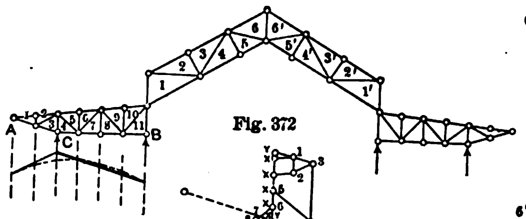


Fig. 372

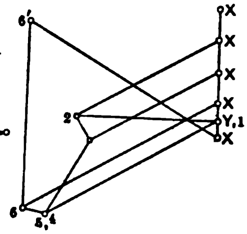


Fig. 373

Fig. 374

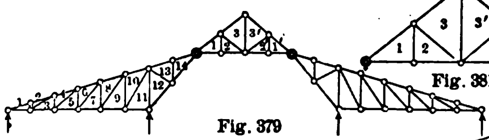


Fig. 379

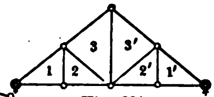


Fig. 381

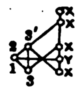
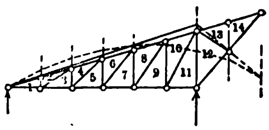


Fig. 382



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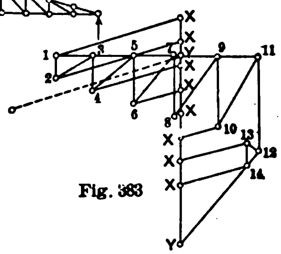

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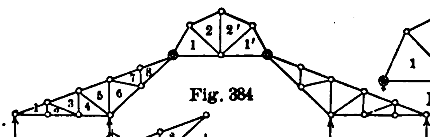

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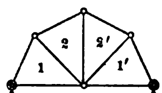


Fig. 388

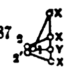


Fig. 387

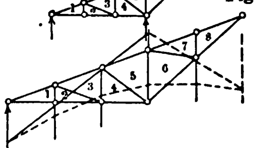


Fig. 386

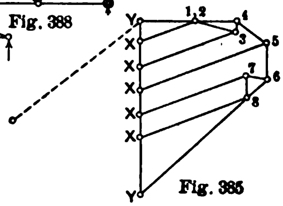


Fig. 385

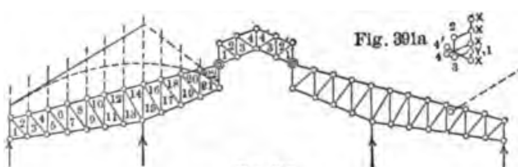


Fig. 390

Fig. 391a

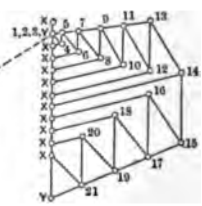
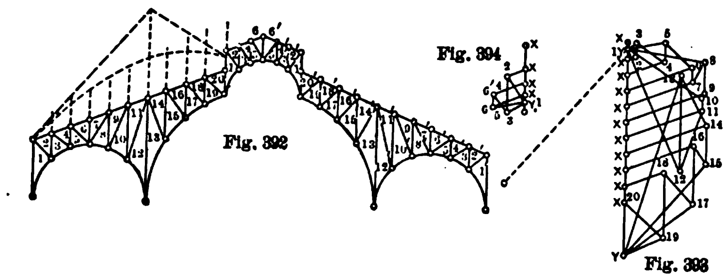


Fig. 391

for the monitor skylight. This truss was worked out in Example 23 of Chapter IV.

Fig. 384 is a similar type with changed form of skylight, with glazed sides and roof. Figs. 386 and 385 are the truss and stress diagrams for the left-hand lower portion of the truss; Figs. 388 and 387 are those for the monitor.

Fig. 390 is a truss of similar purpose but of varied form, with stress diagrams in Figs. 391 and 391 *a*.



FIGS. 392–394

Fig. 393 is a truss on the same principle, but with circular lower chord. The stress diagram for lower left-hand portion is given in Fig. 393, while Fig. 394 is half the stress diagram for the central monitor.

CHAPTER VI

STRESSES BY METHOD OF MOMENTS

274. Origin and Use of Method.—This method for determining the nature and magnitude of the stresses acting in any member of a roof truss is frequently termed Ritter's method, because it was thoroughly developed and applied to most forms of bridge and roof trusses by Dr. August Ritter, formerly of the Polytechnic School at Aix-la-Chapelle, Prussia. An English translation was made by Lieut. H. R. Sankey, R.E., and published in London in 1879.

The method was frequently employed before that of graphostatics became generally known to students and practitioners, and it is still very useful for checking the results obtained by the latter. But it requires more time and labor, and errors are not so readily detected.

275. Diagram and Calculations.—The truss diagram must generally be carefully drawn at a large scale in order to obtain by measurement the lengths of the lever arms of the moments acting at any apex of the truss and producing stresses in its members. The necessary calculations are most easily made with a good 20-inch slide rule, or by means of four or five-place logarithmic tables.

A single example of its complete application to a roof truss will sufficiently exhibit the method of moments, which may often be used with advantage for checking by independently determining the stress occurring in a member.

276. Rules for Proper Application of Method.—Certain rules are to be carefully observed in applying this method.

1. Not more than three members, in which act stresses of unknown magnitudes, may be cut in any panel at one time, since the problem becomes indeterminate in case of four or more such members.

2. The pivot or centre of rotation of the moments of the unknown stresses must be taken at the intersection of the centre lines of two of these three members with unknown stresses. Then since the stress lines in these two members pass through the pivot, each of their moments = 0, leaving but the moment of a single unknown stress, making it possible to compute this moment, afterwards determining the nature (compression or tension) and the magnitude of this un-

known stress, since only one unknown quantity remains in the equation of moments.

3. Since the moments of all forces acting at the cut section of a panel must be in equilibrium with the moment of stress in a member, its equation of moments is to be written and equated to 0.

4. The nature and magnitude of the required stress are then easily found by computation, solving this equation for the stress in the member.

5. The unknown stresses in the members cut in any panel must always be assumed to act outwards from the portion of the truss considered, towards the main portion of the truss.

6. Moments producing watchwise rotation are positive or +; non-watchwise moments are negative or −.

277. Determination of Nature of Stresses.—7. The + sign of the stress in a cut member denotes that the member is in tension; the − sign denotes longitudinal compression in it.

Therefore this method of moments completely determines the nature and magnitudes of the stresses acting in all members of the truss, if there are no superfluous members, *i.e.*, if the truss be entirely composed of triangles. The results of the application of the method should be identical with those obtained by the graphical method.

Example 25.—A Truss with Vertical Ties and Cambered Lower Chord.

278. Programme.—Type as in Fig. 395; span, 100 ft.; rise of upper chord, 20 ft.; of lower chord, 1.5 ft.; 10 panels; trusses 15 ft. on centres; materials, long-leaf pine and steel trusses; covered with tin on 7/8-inch sheathing, wooden rafters and purlins; location at Omaha, about 41° north latitude; medium exposure.

279. Dimensions.—

Inclination $i = 21.8°$.

$$l = \frac{100}{10} = 10.00 \text{ ft.}; \quad l' = \frac{10.00}{\cos 21.8°} = 10.77 \text{ ft.}$$

$$A = 10.77 \times 15.00 = 161.55 \text{ sq. ft.}$$

280. Apex Loads.—

$$\text{Truss} = \frac{100}{25} + \frac{100^2}{12600} = 4.794 \text{ lbs. per horizontal sq. ft.}$$

$$\text{Snow} = 2.5\,(41° - 35°) = 15.00 \text{ lbs. per horizontal sq. ft.}$$

$$\text{Wind} = \frac{8}{9} \times 21.8° = 19.36 \text{ lbs. per inclined sq. ft.}$$

P = 161.55 (2 + 4 + 4 + 3 + 4.794 cos 21.8°) = 2820 lbs. =

...ons.

S = 161.55 (15.00 cos 21.8°) = 2250 lbs. = 1.125 tons.

W = 161.55 × 19.36 = 3128 lbs. = 1.564 tons.

Total Load on Half Truss.—

Permanent = 1.410 × 4 1/2 = 6.345 tons = reaction at A.

Snow = 1.125 × 4 1/2 = 5.0625 tons = reaction at A.

Wind = 1.564 × 4 1/2 = 7.038 tons.

Reaction at A for wind loads may be found by equilibrium polygon or computed by method of moments, as explained later (284).

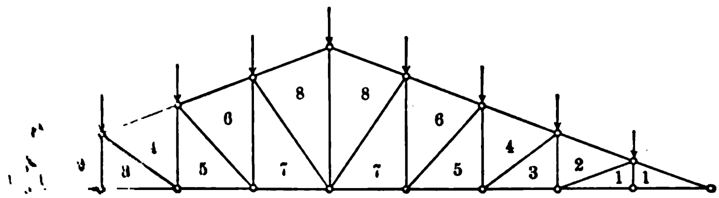


FIG. 395

A. PERMANENT LOADS

Calculation of Permanent Stresses.—Assume that members X 1 and Y 1 are cut from the remainder of the truss as in Fig. 396. The pivot cannot be taken at A, since the moments of the stresses in X 1 and Y 1 about A = 0, and their magnitude cannot be determined. For X 1, assume the pivot b, where a vertical through B cuts Y 1.

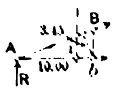


Fig. 396

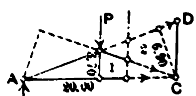


Fig. 397

Member X 1. Fig. 396. Pivot b. Lever arm = 3.45 ft.

Since the portion $A\ B\ b$ cut from the truss must be in equilibrium about the pivot b, write the equation of moments for X 1 as follows, noting that the stress in X 1 produces positive or + rotation about b as well as the reaction at A.

$$0 = + \text{stress } X\,1 \times 3.45 + 6.345 \times 10.00.$$

$$\text{Stress } X\,1 = -\frac{6.345 \times 10.00}{3.45} = -\ 18.39 \text{ tons (compression).}$$

Member Y 1. Fig. 397. Pivot B. Lever arm = 3.70 ft.

$$0 = - \text{stress } Y\,1 \times 3.70 + 6.345 \times 10.00.$$

$$\text{Stress } Y\,1 = +\frac{6.345 \times 10.00}{3.70} = +\ 17.16 \text{ tons (tension).}$$

Member X 2. Fig. 397. Pivot C. Lever arm = 6.90 ft.

$$0 = + \text{stress } X\,2 \times 6.90 + 6.345 \times 20.00.$$

$$\text{Stress } X\,2 = -\frac{6.345 \times 20.00}{6.90} = -\ 16.20 \text{ tons.}$$

Member 1 2. Fig. 397. Pivot A. Lever arm = 7.05 ft.

$$0 = + \text{ stress } 1\,2 \times 7.05 + 1.410 \times 10.00.$$

$$\text{Stress } 1\ 2 = -\frac{1.410 \times 10.00}{7.05} = -\ 2.00 \text{ tons.}$$

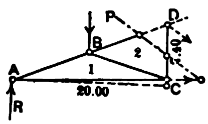


Fig. 398.

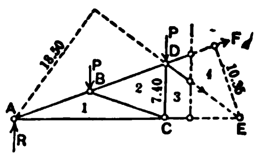


Fig. 399.

Member 2 3. Fig. 398. Pivot A. Lever arm = 20.00 ft.

$$0 = - \text{stress } 2\,3 \times 20.00 + 1.410 \times 10.00.$$

$$\text{Stress } 2\ 3 = +\frac{1.410 \times 10.00}{20.00} = +\ 0.75 \text{ ton.}$$

Member X 4. Fig. 399. Pivot E. Lever arm = 10.35 ft.

$$0 = + \text{stress } X\,4 \times 10.35 + 6.345 \times 30.00 - 1.410 \times (10.00+20.00).$$

$$\text{Stress } X\,4 = -\frac{6.345 \times 30.00 - 1.410 \times 30.00}{10.35} = -\ 14.31 \text{ tons}$$

Member Y 3. Fig. 399. Pivot D. Lever arm = 7.40 ft.

$$0 = - \text{stress } Y\,3 \times 7.40 + 6.345 \times 20 - 1.410 \times 10.$$

$$\text{Stress } Y\,3 = +\frac{6.345 \times 20.00 - 1.41 \times 10.00}{7.40} = +\ 15.24 \text{ tons.}$$

Member 3 4. Fig. 399. Pivot A. Lever arm = 18.15 ft.

$$0 = + \text{stress } 3\,4 \times 18.15 + 1.41\,(10. + 20).$$

10

$$3\ 4 = -\frac{1.41 \times 30.00}{18.15} = -\ 2.33 \text{ tons.}$$

r 4 5. Fig. 400. Pivot A. Lever arm = 30. ft.

$0 = -$ stress 4 5 + 30. + 1.41 (10. +20.00).

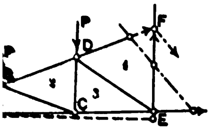


Fig. 400

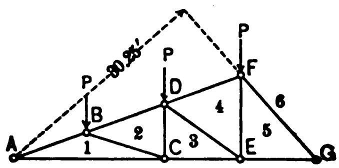


Fig. 401.

$$4\ 5 = +\frac{1.41 \times 30.00}{30.00} = +\ 1.41 \text{ tons.}$$

r X 6. Fig. 401. Pivot G. Lever arm = 13.77 ft.

$0 = +$ stress X 6 $\times$ 13.77 + 6.345 $\times$ 40.$-$ 1.41 (10.+ 20. + 30.).

$$X\ 6 = -\frac{6.345 \times 40.00 - 1.41 \times 60.00}{13.77} = -\ 12.26 \text{ tons.}$$

r Y 5. Fig. 401. Pivot F. Lever arm = 11.1 ft.

$0 = -$ stress Y 5 $\times$ 11.1 + 6.345 $\times$ 30.$-$1.41 (10.+ 20.).

$$Y\ 5 = +\frac{6.345 \times 30.00 - 1.41 \times 30.00}{1.11} = +\ 13.34 \text{ tons.}$$

r 5 6. Fig. 401. Pivot A. Lever arm = 30.25 ft.

$0 = +$ stress 5 6 $\times$ 30.25 + 1.41 (10. + 20. + 30.).

$$5\ 6 = +\frac{1.41 \times 60.00}{30.25} - +\ 2.80 \text{ tons.}$$

r 6 7. Fig. 402. Pivot A. Lever arm = 40. ft.

$0 = -$ stress 6 7 $\times$ 40. + 1.41 (10. + 20. + 30.).

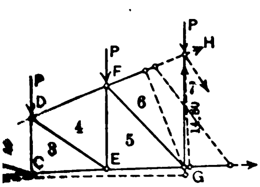


Fig. 402

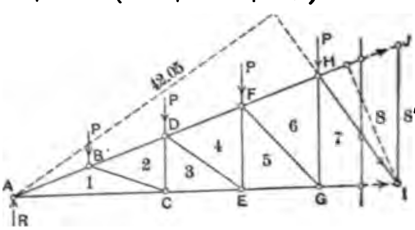


Fig. 403

$$6\ 7 = +\frac{1.41 \times 60.}{40.} = +\ 2.12 \text{ tons.}$$

Member X 8. Fig. 403. Pivot I. Lever arm = 17.2 ft.

$$0 = + \text{stress } X\ 8 \times 17.2 + 6.345 \times 50. - 1.41\ (10 + 20 + 30 + 40).$$

$$\text{Stress } X\ 8 = - \frac{6.345 \times 50.00 - 1.41 \times 100.}{17.2} = - 10.25 \text{ tons.}$$

Member Y 7. Fig. 402. Pivot H. Lever arm = 15.8 ft.

$$0 = - \text{stress } Y\ 7 \times 14.8 + 6.345 \times 40. - 1.41\ (10 + 20 + 30.).$$

$$\text{Stress } Y\ 7 = + \frac{6.345 \times 40. -- 1.41 \times 60.}{14.8} = + 11.43 \text{ tons.}$$

Member 7 8. Fig. 403. Pivot A. Lever arm = 42.05 ft.

$$0 = + \text{stress } 7\ 8 \times 42.05 + 1.41\ (10 + 20 + 30 + 40).$$

$$\text{Stress } 7\ 8 = - \frac{1.41 \times 100.00}{42.05} = - 3.36 \text{ tons.}$$

Member 8 8′. Fig. 403. Pivot A. Lever arm = 50 ft.

$$0 = - \text{stress } 8\ 8' \times 50. \times 1.41\ (10 + 20 + 30 + 40).$$

$$\text{Stress } 8\ 8' = + \frac{1.41 \times 100.}{50.} = + 2.82 \text{ tons.}$$

Note that the stress in 8 8′ must be doubled to provide for stresses coming from both sides of the truss. Hence 8 8′ = 5.64 tons.

B.—Snow Loads

283. Calculation of Snow Stresses.—The snow stresses in members are most readily obtained by computation from permanent stresses by the proportion:

1.410 : 1.125 : : permanent : snow stress in member.

C.—Wind Loads

284. Determination of Windward Reaction.—Sum of reactions at supports = 7.038 tons = total wind load supported by the truss. To obtain reaction at A, write the equation of moments about the other support K.

Reaction at A. Fig. 404. Pivot K. Lever arm = 92.8 ft.

$$0 = + \text{reaction} \times 92.8 - 1.56\left(82.0 + 71.35 + 60.5 + 49.7 + \frac{38.9}{2}\right)$$

$$\text{Reaction} = +\frac{1.564 \times 283.0}{92.8} = +4.77 \text{ tons (upward).}$$

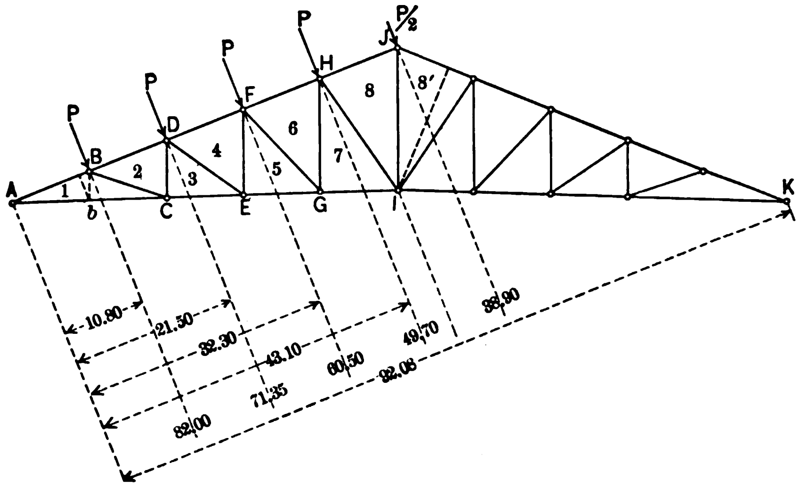


Fig. 404.—Wind Resultants

285. Calculation of Wind Stresses.—Member X 1. Fig. 405. Pivot b. Lever arm = 3.45 ft.

$$0 = +\text{ stress } X\,1 \times 3.45 + 4.77 \times 9.40.$$

$$\text{Stress } X\,1 = -\frac{4.77 \times 9.40}{3.45} = -13.00 \text{ tons.}$$

Member Y 1. Fig. 406. Pivot B. Lever arm = 3.70 ft.

$$0 = +\text{ stress } Y\,1 \times 3.70 + 4.77 \times 10.8.$$

$$\text{Stress } Y\,1 = \frac{4.77 \times 10.8}{3.70} = +13.93 \text{ tons.}$$

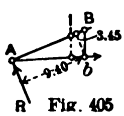


Fig. 405

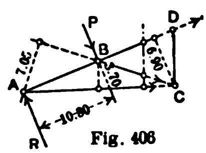


Fig. 406

Member X 2. Fig. 406. Pivot C. Lever arm = 6.90 ft.

$$0 = +\text{ stress } X\,2 \times 6.9 + 4.77 \times 18.82 - 1.564 \times 8.0.$$

$$\text{Stress } X\,2 = -\frac{4.77 \times 18.82 - 1.564 \times 8.0}{6.90} = -11.19 \text{ tons.}$$

Member 1 2. Fig. 406. Pivot A. Lever arm = 7.05 ft.

$$0 = + \text{stress } 1\,2 \times 7.05 + 1.564 \times 10.8.$$

$$\text{Stress } 1\,2 = -\frac{1.564 \times 10.8}{7.05} = -\,2.05 \text{ tons.}$$

Member 2 3. Fig. 407. Pivot A. Lever arm = 20. ft.

$$0 = - \text{stress } 2\,3 \times 20. + 1.564 \times 10.8.$$

$$\text{Stress } 2\,3 = +\frac{1.564 \times 10.8}{20.} = +\,0.85 \text{ ton.}$$

Member X 4. Fig. 408. Pivot E. Lever arm = 10.35 ft.

$$0 = + \text{stress } X\,4 \times 10.35 + 4.77 \times 28.2 - 1.564 \times 24.0.$$

$$\text{Stress } X\,4 = -\frac{4.77 \times 28.2 - 1.564 \times 24.}{10.35} = -\,9.35 \text{ tons.}$$

Member Y 3. Fig. 408. Pivot D. Lever arm = 7.4 ft.

$$0 = - \text{stress } Y\,3 \times 7.4 + 4.77 \times 21.5 - 1.564 \times 10.8.$$

$$\text{Stress } Y\,3. = +\frac{4.77 \times 21.5 - 1.564 \times 10.8}{7.4} = +\,11.57 \text{ tons.}$$

Member 3 4. Fig. 408. Pivot A. Lever arm = 18.15 ft.

$$0 = + \text{stress } 3\,4 \times 18.15 + 1.564\ (10.8 + 21.5).$$

$$\text{Stress } 3\,4 = -\frac{1.564 \times 32.3}{18.15} = -\,2.78 \text{ tons.}$$

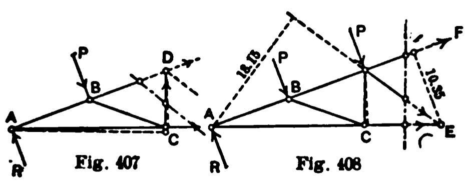


Fig. 407 Fig. 408

Member 4 5. Fig. 409. Pivot A. Lever arm = 30. ft.

$$0 = - \text{stress } 4\,5 \times 30. - 1.564\ (10.8 + 21.5).$$

$$\text{Stress } 4\,5 = +\frac{1.564 \times 32.3}{30.} = +\,1.79 \text{ tons.}$$

Member X 6. Fig. 410. Pivot G. Lever arm = 13.77 ft.

$$0 = + \text{stress } X\,6 \times 13.77 + 4.77 \times 37.6 - 1.564\ (5.3 + 16.1 + 26.8).$$

$$\text{Stress } X\,6 = -\frac{4.77 \times 37.6 - 1.564 \times 48.2}{13.77} = -7.55 \text{ tons.}$$

Member Y 5. Fig. 409. Pivot F. Lever arm = 11.1 ft.

$$0 = -\text{stress } Y\,5 \times 11.1 + 4.77 \times 32.3 - 1.564\,(10.8 + 21.5).$$

$$\text{Stress } Y\,5 = +\frac{4.77 \times 32.3 - 1.564 \times 32.3}{11.1} = +9.32 \text{ tons.}$$

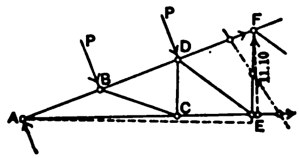


Fig. 409

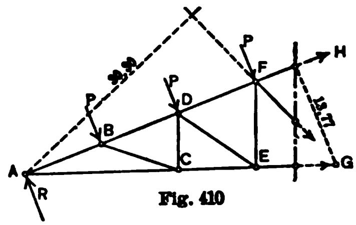


Fig. 410

Member 5 6. Fig. 410. Pivot A. Lever arm = 31.25 ft.

$$0 = +\text{stress } 5\,6 \times 30.25 + 1.564\,(10.8 + 21.5 + 32.3).$$

$$\text{Stress } 5\,6 = -\frac{1.564 \times 64.6}{30.25} = -3.34 \text{ tons.}$$

Member 6 7. Fig. 411. Pivot A. Lever arm = 30 ft.

$$0 = +\text{stress } 6\,7 \times 30. + 1.564\,(10.8 + 21.5 + 32.3).$$

$$\text{Stress } 6\,7 = +\frac{1.564 \times 64.6}{30.0} = +3.37 \text{ tons.}$$

Member X 8. Fig. 412. Pivot I. Lever arm = 17.2 ft.

$$0 = -\text{stress } X\,8 \times 17.2 + 4.77 \times 47.0 - 1.564\,(3.9 + 14.7 + 25.4 + 36.2).$$

$$\text{Stress } X\,8 = -\frac{4.77 \times 47.0 - 1.564 \times 80.2}{17.2} = -5.72 \text{ tons.}$$

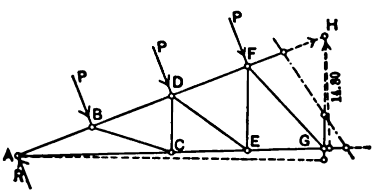


Fig. 411

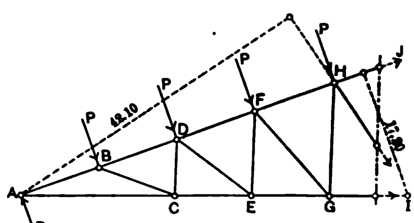


Fig. 412

Member Y 7. Fig. 411. Pivot H. Lever arm = 14.8 ft.

$$0 = -\text{stress } Y\,7 \times 14.8 + 4.77 \times 43.1 - 1.564\,(10.8 + 21.6 + 32.3).$$

$$\text{Stress } Y\,7 = + \frac{4.77 \times 43.1 - 1.564 \times 64.7}{14.8} = + 7.15 \text{ tons.}$$

Member 7 8. Fig. 412. Pivot A. Lever arm = 42.1 ft.

$$0 = + \text{stress } 7\,8 \times 42.1 + 1.564\,(10.8 + 21.6 + 32.3 + 43.1).$$

$$\text{Stress } 7\,8 = - \frac{1.564 \times 107.8}{42.1} = - 4.01 \text{ tons.}$$

Member 8 8′. Fig. 412. Pivot A. Lever arm = 50. ft.

$$0 = - \text{stress } E\,8' \times 50. + 1.564\,(10.8 + 21.5 + 32.3 + 42.1).$$

$$\text{Stress } 8\,8' = + \frac{1.564 \times 106.7}{50.} = + 3.34 \text{ tons.}$$

The stress in member $E\,8'$ caused by the wind load is not to be doubled.

Reaction at K = 7.038 − 4.77 = 2.268 tons.

Member $X\,8'$. Fig. 404. Pivot L. Lever arm = 17.1 ft.

$$0 = + \text{stress } X\,8' \times 17.1 - (- 2.268 \times 44.5).$$

$$\text{Stress } X\,8' = - \frac{2.268 \times 44.5}{17.1} = - 5.9 \text{ tons.}$$

Since this is larger than the stress X 8 already found, it is to be placed in the stress sheet instead of stress X 8.

286. Stress Sheet for Example 25.—

Member.	P-stress.	S-stress.	W-stress.	Maximum.	Minimum.
X 1	−18.4	−14.7	−13.0	−46.1	−18.4
X 2	−16.2	−12.9	−11.2	−40.3	−16.2
X 4	−14.3	−11.4	− 9.4	−35.1	−14.3
X 6	−12.3	− 9.8	− 7.6	−29.7	−12.3
X 8	−10.3	− 8.2	− 5.9	−24.4	−10.3
Y 1	+17.2	+13.7	+13.9	+44.8	+17.2
Y 3	+15.2	+12.1	+11.6	+38.9	+15.2
Y 5	+13.3	+10.6	+ 9.3	+33.2	+13.3
Y 7	+11.4	+ 9.1	+ 7.2	+27.7	+11.4
1 2	− 2.0	− 1.6	− 2.1	− 5.7	− 2.0
3 4	− 2.3	− 1.8	− 2.8	− 6.9	− 2.3
5 6	− 2.8	− 2.2	− 3.3	− 8.3	− 2.8
7 8	− 3.4	− 2.7	− 4.0	−10.1	− 3.4
2 3	+ 0.7	+ 0.6	+ 0.9	+ 2.2	+ 0.7
4 5	+ 1.4	+ 1.1	+ 1.8	+ 4.3	+ 1.4
6 7	+ 2.1	+ 1.7	+ 3.4	+ 7.2	+ 2.1
8 8′	+ 2.8	+ 2.2	+ 3.4	+ 8.4	+ 2.8

CHAPTER VII

LENGTHS OF MEMBERS OF ROOF TRUSSES

287. Accuracy Required.—Before commencing to detail the connections of members at the apexes of the truss, it becomes necessary to compute accurately the centre length of each member between the centres of the apexes joined by it. This length is required in feet, inches, and fractions, usually to the nearest $\frac{1}{32}$ inch, which corresponds to $\frac{1}{384}$, or nearly $\frac{1}{400}$ foot.

288. Aids in Computations.—These computations are greatly facilitated and the liability to error is lessened, if a good table of squares and logarithms is employed. Everything necessary for computing lengths not exceeding 100 ft. is contained in Smoley's tables of squares, logarithms, trigonometric functions, etc. For greater lengths or when more minute accuracy is required, Bremiker's six-place tables or Schron's or Vega's seven-place tables of logarithms should be used.

289. Algebraic Formulas.—Algebraic formulas might be given for the lengths of members (Ricker's "Trussed Roofs," Chapter V), but for nearly all forms of trusses in actual use, these lengths may be readily computed by a proper application of the properties of the right-angled triangle. It is further certain that a series of computed examples will be of much greater assistance than mere algebraic formulas, especially in practice. Hence the latter method is here employed, care being taken to explain any unusual or difficult computations.

290. Dimensions Assumed for Examples.—For convenience and to avoid unnecessary repetition, the following dimensions are assumed in all the following examples, unless otherwise stated.

Span 100 ft.; rise of upper chord 20 ft.; rise of lower chord 0 or 2.5 ft.; truss usually divided into 10 panels of equal horizontal length, or 8 panels for Fink truss; trusses with circular chords are variously subdivided into panels, as stated in each case.

EXAMPLE 1.—A TRIANGULAR TRUSS. (Fig. 413.)

Same as Example 1 of Chapter IV.

291. Computations.—

Vertical 9 9′ = 20′ 0″ by construction.

$$\text{Vertical } 7\ 8 = 20' \times \frac{4}{5} = 16'\ 0''.$$

$$\text{Vertical } 5\ 6 = 20' \times \frac{3}{5} = 12'\ 0''.$$

$$\text{Vertical } 3\ 4 = 20' \times \frac{2}{5} = 8'\ 0''.$$

$$\text{Vertical } 1\ 2 = 20' \times \frac{1}{5} = 4'\ 0''.$$

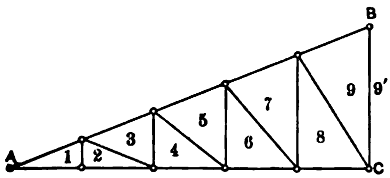


FIG. 413

$Y\,1 = Y\,2 = Y\,4 = Y\,6 = Y\,8 = 10'\ 0''$ each by construction.

$$X\,1 = X\,3 = X\,5 = X\,7 = X\,9 = \sqrt{10^2 + 4^2} = \sqrt{116} = 10'\ 9\frac{1}{4}''.$$

$$\text{Principal } A\,B = 5\left(10'\ 9\frac{1}{4}''\right) = 53'\ 10\frac{1}{4}''.$$

$$\text{Diagonal } 2\ 3 = \sqrt{10^2 + 4^2} = 10'\ 9\frac{1}{4}''.$$

$$\text{Diagonal } 4\ 5 = \sqrt{10^2 + 8^2} = 12'\ 9\frac{11}{16}''.$$

$$\text{Diagonal } 6\ 7 = \sqrt{10^2 + 12^2} = 15'\ 7\frac{7}{16}''.$$

$$\text{Diagonal } 8\ 9 = \sqrt{10^2 + 16^2} = 18'\ 10\frac{13}{32}''.$$

EXAMPLE 2.—SAME TRUSS WITH REVERSED DIAGONALS (Fig. 414)

Same as Example 2 of Chapter IV.

Lengths of all members except diagonals are unchanged.

292. Computations.—

$$\text{Diagonal } 2\ 3 = \sqrt{10^2 + 8^2} = 12'\ 9\frac{11}{16}''.$$

$$\text{Diagonal } 4\ 5 = \sqrt{10^2 + 12^2} = 15'\ 7\frac{7}{16}''.$$

$$\text{Diagonal } 6\ 7 = \sqrt{10^2 + 16^2} = 18'\ 10\frac{13}{32}''.$$

$$\text{Diagonal } 8\ 9 = \sqrt{10^2 + 20^2} = 22'\ 4\frac{11}{32}''.$$

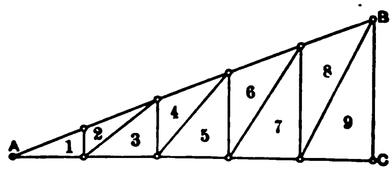


Fig. 414

Example 3.—Same Truss as in Example 1, Excepting that the Lower Chord is Cambered 2′ 6″ at Middle (Fig 415)

293. Computations.—

$$\text{Vertical } 9\ 9' = 20'\ 0'' - 2'\ 6'' = 17'\ 6''.$$

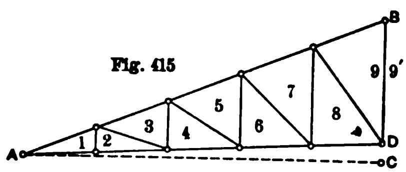
$$\text{Vertical } 7\ 8 = \frac{4}{5}\ (17'\ 6'') = 14'\ 0''.$$

$$\text{Vertical } 5\ 6 = \frac{3}{5}\ (17'\ 6'') = 10'\ 6''.$$

$$\text{Vertical } 3\ 4 = \frac{2}{5}\ (17'\ 6'') = 7'\ 0''.$$

$$\text{Vertical } 1\ 2 = \frac{1}{5}\ (17'\ 6'') = 3'\ 6''.$$

Fig. 415



Difference in heights of lower ends of adjacent verticals $= \dfrac{2'\ 6''}{5} =$ 0′ 6″, which is likewise the rise of y 1.

$$Y1 = Y2 = Y4 = Y6 = Y8 = \sqrt{10^2 + 0.5^2} = \sqrt{100.25} = 10'\ 0\frac{5}{32}''.$$

For computing diagonals, the altitude of each right-angled triangle = corresponding vertical − 0.5′.

$$\text{Diagonal } 2\ 3 = \sqrt{10^2 + (3.5 - 0.5)^2} = \sqrt{109} = 10'\ 5\frac{5}{16}''.$$

$$\text{Diagonal } 4\ 5 = \sqrt{10^2 + (7.0 - 0.5)^2} = \sqrt{142.25} = 11'\ 11\frac{1}{8}''.$$

$$\text{Diagonal } 6\ 7 = \sqrt{10^2 + (10.5 - 0.5)^2} = \sqrt{200} = 14'\ 1\frac{23}{32}''.$$

$$\text{Diagonal } 8\ 9 = \sqrt{10^2 + (14.0 - 0.5)^2} = \sqrt{282.25} = 16'\ 9\frac{19}{32}''.$$

Example 4.—Same Truss with Reversed Diagonals (Fig. 416)

The lengths of the diagonals are alone changed.

294. Computations.—For computing diagonals, the altitude of each right-angled triangle = corresponding vertical + 0.5′.

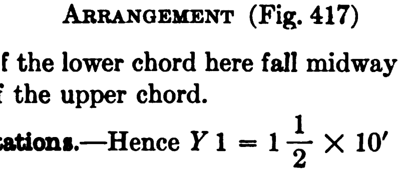
$$\text{Diagonal } 2\ 3 = \sqrt{10^2 + (7.0 + 0.5)^2} = \sqrt{156.25} = 12'\ 6''.$$

$$\text{Diagonal } 4\ 5 = \sqrt{10^2 + (10.5 + 0.5)^2} = \sqrt{221} = 14'\ 10\frac{13}{32}''.$$

$$\text{Diagonal } 6\ 7 = \sqrt{10^2 + (14.0 + 0.5)^2} = \sqrt{310.25} = 17'\ 7\frac{3}{8}''.$$

$$\text{Diagonal } 8\ 9 = \sqrt{10^2 + (17.5 + 0.5)^2} = \sqrt{424} = 20'\ 7\frac{3}{32}''.$$

Fig. 416



Example 5.—Truss with Web Members of Howe Type of Arrangement (Fig. 417)

The apexes of the lower chord here fall midway between verticals through those of the upper chord.

295. Computations.—Hence $Y\,1 = 1\frac{1}{2} \times 10' = 15'\ 0''$.

There are no verticals in the truss, but lengths of dotted verticals are computed in the same manner as in Example 3, and they have the same lengths in this case.

$$\text{Diagonal } 1\ 2 = \sqrt{5^2 + 4^2} = \sqrt{41} = 6'\ 4\frac{27}{32}''.$$

Diagonals 2 3 and 3 4 $= \sqrt{5^2 + 8^2} = \sqrt{89} = 9'\,5\frac{7}{32}''$.

Diagonals 4 5 and 5 6 $= \sqrt{5^2 + 12^2} = \sqrt{169} = 13'\,0''$.

Diagonals 6 7 and 7 8 $= \sqrt{5^2 + 16^2} = \sqrt{281} = 16'\,9\frac{5}{32}''$.

Diagonal 8 9 $= \sqrt{5^2 + 20^2} = \sqrt{425} = 20'\,7\frac{3}{8}''$.

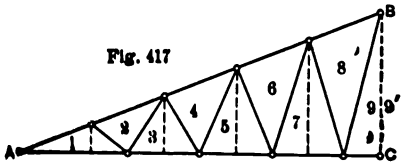


Fig. 417

EXAMPLE 6. SAME TRUSS WITH LOWER CHORD CAMBERED 2′ 6″ (Fig. 418)

Lengths of dotted verticals through apexes of upper chord are here the same as in Example 3, Fig. 415.

296. Computations.—Lengths of X members are as in Example 5, Fig. 417.

Lengths of Y members are as in Example 4, Fig. 416.

Excepting for Y 1, which $= 1\frac{1}{2}\ (10'\,0\frac{5}{32}'') = 15'\,0\frac{1}{4}''$.

And for Y 9, which $= \frac{1}{2}\ (10'\,0\frac{5}{32}'') = 5'\,0\frac{3}{32}''$.

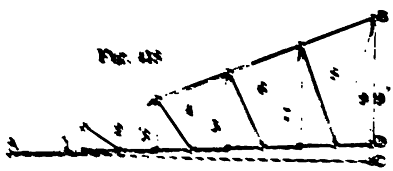


Fig. 418

For computing diagonals, altitude of corresponding right-angled triangle = length of corresponding vertical − 0.25′.

Diagonal 1 2 $= \sqrt{5^2 + (3.5 - 0.25)^2} = \sqrt{35.5625} = 5'\,11\frac{9}{16}''$.

Diagonal 2 3 $= \sqrt{5^2 + (7.5 - 0.25)^2} = \sqrt{77.5625} = 8'\,9\frac{11}{16}''$.

Diagonal 3 4 $= \sqrt{5^2 + (7.0 - 0.25)^2} = \sqrt{70.5625} = 8'\ 4\frac{13}{16}''$.

Diagonal 4 5 $= \sqrt{5^2 + (10.5 + 0.25)^2} = \sqrt{140.565} = 11'\ 10\frac{9}{32}''$.

Diagonal 5 6 $= \sqrt{5^2 + (10.5 - 0.25)^2} = \sqrt{130.065} = 11'\ 4\frac{27}{32}''$.

Diagonal 6 7 $= \sqrt{5^2 + (14.0 + 0.25)^2} = \sqrt{228.065} = 15'\ 1\frac{1}{32}''$.

Diagonal 7 8 $= \sqrt{5^2 + (14.0 - 0.25)^2} = \sqrt{214.065} = 14'\ 7\frac{9}{16}''$.

Diagonal 8 9 $= \sqrt{5^2 + (17.5 + 0.25)^2} = \sqrt{340.065} = 18'\ 5\frac{9}{32}''$.

Example 7.—Fink Truss with 8 Panels (Fig. 419)

Same as Example 3 of Chapter IV.

The true Fink truss is divided into 4, 8, 16, or 32 panels; the lower chord may be horizontal, cambered, or raised.

297. Computations.—

Length of the principal is here $53'\ 10\frac{1}{4}''$ as before.

Half length of principal $= \dfrac{53'\ 10\ 1/4''}{2} = 26'\ 11\frac{1}{8}''$.

$$X\ 1 = \frac{53'\ 10\ 1/4''}{4} = 13'\ 5\frac{9}{16}''.$$

Then 3 4 $= (26'\ 11\frac{1}{8}'')\tan 21°\ 48'\ 5'' = 10'\ 9\frac{1}{4}''$.

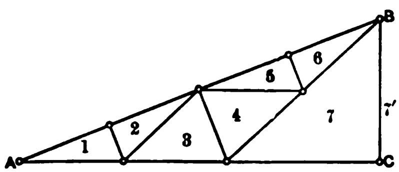


Fig. 419

$$1\ 2 = 5\ 6 = \frac{10'\ 9\ 1/4''}{2} = 5'\ 4\frac{5}{8}''.$$

$Y\ 1 = Y\ 3 = 2\ 3 = 4\ 5 = 4\ 7 = 6\ 7 = \sqrt{(13'\ 5\ 9/16'')^2 + (5'\ 4\ 5/8'')^2} =$ $14'\ 6''$.

$Y\ 7' = 50'\ 0'' - 2\ (14'\ 6'') = 21'\ 0''$.

EXAMPLE 8.—SAME TRUSS WITH LOWER CHORD CAMBERED 2′ 6″ (Fig. 420)

Evidently the lengths of principal and X members are as in Example 7.

298. Computations.—

Tan i or $B\ A\ C$ = tan 21° 48′ 5″ as before.

Tan i'' or $D\ A\ C$ = tan 2° 51′ 45″.

Then $i'' = B\ A\ D$ = (21° 48′ 5″) − (2° 51′ 45″) = 18° 56′ 20″.

$$3\,4 = (26'\,11\,1/8'') \tan 18°\,56'\,20'' = 9'\,2\frac{7}{8}''.$$

$$1\,2 = 5\,6 = \frac{9'\,2\,7/8''}{2} = 4'\,7\frac{7}{16}''.$$

$$Y\,1 = Y\,3 = 2\,3 = 4\,5 = 4\,7 = 6\,7 =$$

$$\sqrt{(13'\,5\,9/16'')^2 + (4'\,7\,7/16'')^2} = 14'\,2\frac{3}{8}''.$$

Half lower chord $= \sqrt{50^2 + 2.5^2} = \sqrt{2506.25} = 50'\,0\frac{3}{4}''.$

$Y\,7 = (50'\,0\frac{3}{4}'') - 2(14'\,2\frac{3}{8}'') = 21'\,8''.$

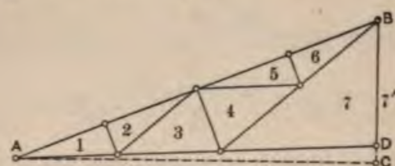


FIG. 420

EXAMPLE 9.—SAME TRUSS WITH LOWER CHORD RAISED 2′ 6″ (Fig. 421)

299. Computation.—Angle $B\ A\ C$ = 21° 45′ 5″ as before.

Perpendicular $F\ H = (26'\,11\frac{1}{8}'') \tan 21°\,48'\,5'' = 10'\,9\frac{1}{4}''$, as in Example 7.

Draw $F\ D$ horizontal and bisecting rise of upper chord at D.

Since $G\ E$ is also horizontal by construction, it divides the perpendicular $F\ H$ at G in the same ratio as it divides the vertical DC at E. Hence $DC : DE :: FH : F\,G$. Or $10 : 10 - 2.5 :: 10'\,9\frac{1}{4}''$:

$$\text{length of member } 3\ 4 = \frac{(10'\ 9\ 1/4'') \times (7'\ 6'')}{10'\ 6''} = 8'\ 0\frac{15}{16}''.$$

$$1\ 2 = 5\ 6 = \frac{8'\ 0\ 15/16''}{2} = 4'\ 0\frac{15}{32}''.$$

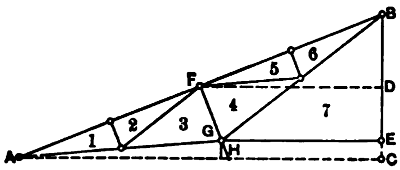


FIG. 421

$$Y\,1 = Y\,3 = 2\,3 = 4\,5 = 4\,7 = 6\,7 =$$

$$\sqrt{(13'\ 5\ 9/16'')^2 + (4'\ 0\ 15/32'')^2} = 14'\ 0\frac{11}{16}''.$$

$$Y\,1 + Y\,3 = 2\,(14'\ 0\frac{11}{16}'') = 28'\ 1\frac{3}{8}''.$$

Horizontal projection of $(Y\,1 + Y\,3) =$

$$\sqrt{(28'\ 1\ 3/8'')^2 - (2'\ 6'')^2} = 28'\ 0\frac{1}{32}''.$$

Hence $Y\,7 = 50'\ 0'' - 28'\ 0\frac{1}{32}'' = 21'\ 11\frac{31}{32}''.$

Vertical $7\ 7' = 20'\ 0'' - 2'\ 6'' = 17'\ 6''.$

EXAMPLE 10.—MODIFIED FINK TRUSS WITH 10 PANELS (Fig. 422)

Same as Example 6 of Chapter IV. It is sometimes advisable to use a different number of panels while retaining the Fink principle as far as possible.

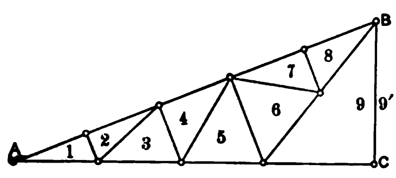


FIG. 422

300. Computations.—$X\,1$ here $= 10'\ 9\frac{1}{4}''$ as in Example 1, etc.

$$X\,1 + X\,2 + X\,4 = 3\,(10'\ 9\frac{1}{4}'') = 32'\ 3\frac{3}{4}''.$$

Angle $B\,A\,C = 21°\ 48'\ 5''$ as before.

$$5\,6 = (32'\ 3\frac{3}{4}'')\ \tan 21°\ 48'\ 5'' = 12'\ 11\frac{3}{32}''.$$

$$1\,2 = \frac{12'\ 11\ 3/32''}{3} = 4'\ 3\frac{23}{32}''.$$

$$3\,4 = \frac{2}{3}\,(12'\ 11\frac{3}{32}'') = 8'\ 7\frac{3}{8}''.$$

$$7\,8 = \frac{12'\ 11\ 3/32''}{2} = 6'\ 5\frac{9}{16}''.$$

$$Y\,1 = Y\,3 = Y\,5 = 2\,3 = \sqrt{(10'\ 9\ 1/4'')^2 + (4'\ 3\ 23/32'')^2} = 11'\ 7\frac{3}{16}''.$$

$$4\,5 = \sqrt{(10'\ 9\ 1/4'')^2 + (8'\ 7\ 3/8'')^2} = 13'\ 9\frac{1}{2}''.$$

$$6\,7 = 6\,9 = 8\,9 = \sqrt{(10'\ 9\ 1/4'')^2 + (6'\ 5\ 9/16'')^2} = 12'\ 6\frac{3}{4}''.$$

$$Y\,9 = 50'\ 0'' - 3\,(11'\ 7\frac{3}{16}'') = 15'\ 2\frac{7}{16}''.$$

EXAMPLE 11.—MODIFIED FINK TRUSS WITH LOWER CHORD CAMBERED 2′ 6″ (Fig. 423)

301. Computations.—

Angle $B\,A\,C = 21°\ 48'\ 5''$ as in Example 9.
Angle $D\,A\,C = 2°\ 51'\ 45''$ as in Example 8.
Angle $B\,A\,D = 18°\ 56'\ 20''$ as in same.

$$5\,6 = 3\,(10'\ 9\frac{1}{4}'')\ \tan 18°\ 56'\ 20'' = 11'\ 1\frac{1}{32}''.$$

$$1\,2 = \frac{11'\ 1\ 1/32''}{3} = 3'\ 8\frac{11}{32}''.$$

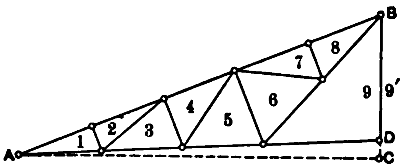


FIG. 423

$$3\,4 = \frac{2}{3}\,(11'\ 1\frac{1}{32}) = 7'\ 4\frac{11}{16}''.$$

$$7\,8 = \frac{11'\ 1\ 1/32''}{2} = 5'\ 6\frac{17}{32}''.$$

$$Y1 = Y3 = Y5 = 2\,3 = \sqrt{(10'\,9\,1/4'')^2 + (3'\,8\,11/32'')^2} = 11'\,4\frac{21}{32}''.$$

$$\text{Half lower chord} = \sqrt{50^2 + 2.5^2} = 50'\,0\frac{3}{4}''.$$

$$Y9 = 50'\,0\frac{3}{4}'' - 3\,(11'\,4\frac{21}{32}'') = 15'\,10\frac{25}{32}''.$$

Example 12.—Modified Fink Truss with Lower Chord Raised 2′ 6″ (Fig. 424)

302. Computations.—Lengths of X members are same as in the last example.

Draw horizontal $F\,D$ through apex F, cutting $B\,C$ at D.

Since $F\,D$ and $G\,E$ are horizontal and parallel, the line $G\,E$ intersects $F\,H$ in the same ratio as $D\,C$ at E.

$$F\,H = 3\,(10'\,9\frac{1}{4}'')\tan 18°\,56'\,20'' = 11'\,1\frac{1}{32}''.$$

Then $D\,C : D\,E :: F\,H :$ member $5\,6$.

$$\text{Or } 12'\,0'' : 9'\,6'' :: 11'\,1\frac{1}{32}'' : 10'\,2\frac{25}{32}''.$$

$$1\,2 = \frac{10'\,2\,25/32''}{3} = 3'\,4\frac{15}{16}''.$$

$$3\,4 = \frac{2}{3}\,(10'\,2\frac{25}{32}'') = 6'\,9\frac{27}{32}''.$$

$$7\,8 = \frac{10'\,2\,25/32''}{2} = 5'\,1\frac{13}{32}''.$$

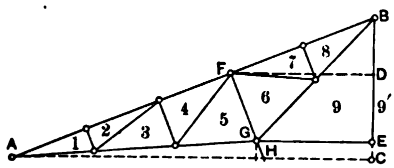


Fig. 424

$$Y1 = Y3 = Y5 = \sqrt{(10'\,9\,1/4'')^2 + (3'\,4\,15/16'')^2} = 11'\,3\frac{19}{32}''.$$

$$Y1 + Y3 + Y5 = 3\,(11'\,3\frac{19}{32}'') = 33'\,10\frac{25}{32}''.$$

Horizontal projection of same =

$$\sqrt{(33'\,10\,25/32'')^2 - 2.5^2} = 33'\,9\frac{11}{16}''.$$

$$Y\,9 = 50'\,0'' - 33'\,9\frac{11}{16}'' = 16'\,2\frac{5}{16}''.$$

EXAMPLE 13.—FINK TRUSS WITH SIDES OF EQUAL SLOPES BUT OF UNEQUAL LENGTHS (Fig. 425)

Same as Example 7 of Chapter IV.

303. Computations.—Span, 120 ft.; left side, 80 ft.; right side, 40 ft.; rise of ridge, 40 ft.; $C\,B = 20$ ft., making an inclined lower chord.

$$\text{Tan}\,P\,A\,E = \frac{40}{80} = \tan 26^\circ\,33'\,54''.$$

$$\text{Tan}\,D\,A\,E = \frac{20}{80} = \tan\ \ 9^\circ\,27'\,44''.$$

$$\text{Hence, } P\,A\,D = 17^\circ\ \ 6'\,10''.$$

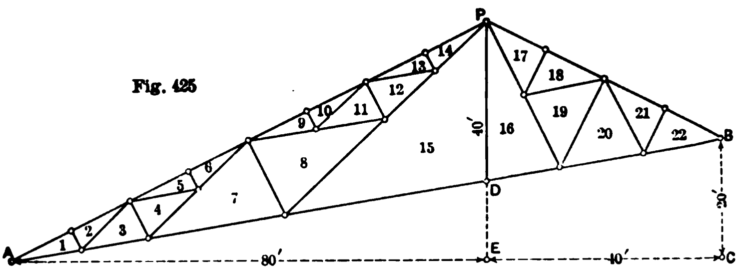
$$\text{And } P\,D\,B = 36^\circ\ \ 1'\,38''.$$

By similar triangles, $A\,C : A\,E :: B\,C : D\,E$.

Or $120' : 80' :: 20' : 13'\,4''$,

Vertical $15\,16 = 40'\,0'' - 13'\,4'' = 26'\,8''$.

Fig. 425



$$\text{Length of lower chord} = \sqrt{120^2 + 20^2} = \sqrt{14400} = 121'\,7\frac{7}{8}''.$$

$$\text{Length half principal } A\,P = \sqrt{40^2 + 20^2} = \sqrt{2000} = 44'\,8\frac{21}{32}''.$$

$$X\,1, \text{etc.} = \frac{44'\,8\,21/32''}{4} = 11'\,2\frac{5}{32}''.$$

$$7\,8 = (44'\,8\frac{21}{32}'')\tan 17^\circ\,6'\,10'' = 13'\,9\frac{1}{8}''.$$

$$3\,4 = 11\,12 = \frac{13'\,9\,1/8''}{2} = 6'\,10\frac{9}{16}''.$$

$$1\,2 = 5\,6 = 9\,10 = 13\,14 = \frac{13'\,9\,1/8''}{4} = 3'\,5\frac{9}{32}''.$$

Also $P\,B = 44'\,8\frac{21}{32}''$.

Half $P\,B = 22'\,4\frac{11}{32}''$.

$19\;20 = (22'\,4\frac{11}{32}'')\tan 36°\,1'\,38'' = 16'\,3\,5\frac{5}{32}''$.

$17\;18 = 21\;22 = \dfrac{16'\,3\,5/32''}{2} = 8'\,1\frac{19}{32}''$.

$Y\,1 = Y\,3$, etc. $= \sqrt{(11'\,2\,5/32'')^2+(3'\,5\,9/32'')^2} = 11'\,8\frac{3}{8}''$.

$Y\,7 = 8\;15 = 2\,(11'\,8\frac{3}{8}'') = 23'\,4\frac{23}{32}''$.

$Y\,22 = Y\,20$, etc. $= \sqrt{(11'\,2\,5/32'')^2+(8'\,1\,19/32'')^2} = 13'\,9\frac{29}{32}''$.

$A\,D = \frac{2}{3}\,(121'\,7\frac{7}{8}'') = 81'\,1\frac{1}{4}''$.

$D\,B = \frac{1}{3}\,(121'\,7\frac{7}{8}'') = 40'\,6\frac{5}{8}''$.

$Y\,15 = 81'\,1\frac{1}{4}'' - 2\,(23'\,4\frac{23}{32}'') = 34'\,3\frac{13}{16}''$.

$Y\,16 = 40'\,6\frac{5}{8}'' - 2\,(13'\,9\frac{29}{32}'') = 12'\,10\frac{13}{16}''$.

Example 14.—Truss with Segmental Upper Chord (Fig. 426)

One-half the entire truss is shown in the figure. Its span is divided into 10 equal horizontal panels, which produces a variable division of the upper chord.

304. Radius of Segmental Arc.—Join $A\,J$ and bisect $A\,J$ at M; draw $M\,L$ perpendicular to $A\,J$; this cuts the extended middle vertical at L, the centre of the segmental arc forming the upper chord. The radius $J\,L$ may likewise be computed, as follows:

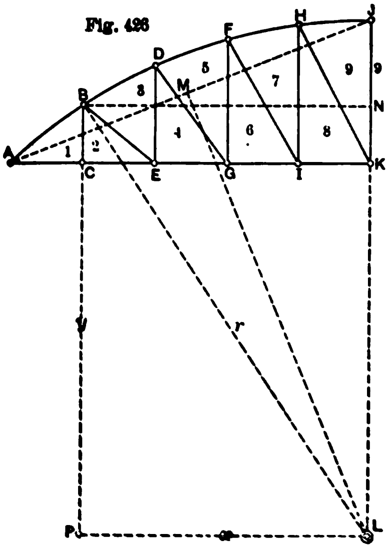
Since the right-angled triangles $A\,J\,K$ and $L\,J\,M$ have a common angle $M\,J\,K$ at J, their corresponding sides must be proportional to each other, and $J\,K : J\,M :: J\,A : J\,L$. Here $A\,K = 50'\;0''$ and $J\,K = 20'\,0''$.

Hence $J\,A = \sqrt{50^2+20^2} = \sqrt{2900} = 53'\,10\frac{1}{4}''$.

$$J\,M = \frac{1}{2}\,(53'\ 10\frac{1}{4}'') = 26'\ 11\frac{1}{8}''.$$

Then $20'\ 0'' : 26'\ 11\frac{1}{8}'' :: 53'\ 10\frac{1}{4}'' : 72'\ 6\frac{1}{16}'' =$ radius $J\,L$.

Fig. 426



305. Computations.—Join apex B with centre L; draw horizontals $B\,N$ and $P\,L$ and vertical $B\,P$. Since the equation of the circle with centre L at origin of coordinates is:

$$r^2 = x^2 + y^2,$$

then ordinate $y = \sqrt{r^2 - x^2}$.

Then $B\,P = \sqrt{B\,L^2 - B\,N^2} = \sqrt{(72'\ 6\ 1/16'')^2 - 40^2} = \sqrt{3657.} = 60'\ 5\frac{11}{16}$

Now $C\,P = K\,L = 72'\ 6\frac{1}{16}'' - 20'\ 0'' = 52'\ 6\frac{1}{16}''$.

Hence vertical 1 2 $= 60'\ 5\frac{11}{16}'' - 52'\ 6\ 1\frac{1}{16}'' = 7'\ 11\frac{5}{8}''$.

Also

Vertical 3 4 $= \sqrt{(72'\ 6\ 1/16'')^2 - 30^2} - 52'\ 6\frac{1}{16}'' = 13'\ 6\frac{1}{32}''$.

Vertical 5 6 $= \sqrt{(72'\ 6\ 1/16'')^2 - 20^2} - 52'\ 6\frac{1}{16}'' = 17'\ 2\frac{1}{4}''$.

Vertical 7 8 $= \sqrt{(72' \, 6\,1/16'')^2 - 10^2} - 52' \, 6\frac{1}{16}'' = 19' \, 3\frac{11}{16}''$.

Vertical 9 9′ by construction = 20′ 0″.

Diagonal 2 3 $= \sqrt{10^2 + (7' \, 11\,5/8'')^2} = \sqrt{163.5010} = 12' \, 9\frac{7}{16}''$.

Diagonal 4 5 $= \sqrt{10^2 + (13' \, 6\,1/32'')^2} = \sqrt{282.3203} = 16' \, 9\frac{5}{8}''$.

Diagonal 6 7 $= \sqrt{10^2 + (17' \, 2\,1/4'')^2} = \sqrt{395.4102} = 19' \, 10\frac{5}{8}''$.

Diagonal 8 9 $= \sqrt{10^2 + (19' \, 3\,11/16'')^2} = \sqrt{472.7715} = 21' \, 8\frac{29}{32}''$.

Evidently $\sqrt{10^2 + (\text{vertical projection of arc panel})^2} =$ length of straight chord panel connecting two adjacent apexes of the upper chord. (Fig. 427.)

These vertical projections = difference between lengths of successive verticals.

Chord $X\,1 = \sqrt{10^2 + (7' \, 11\,5/8'')^2} = \sqrt{163.501} = 12' \, 9\frac{7}{16}''$.

Chord $X\,3 = \sqrt{10^2 + (13' \, 6\,1/32'' - 7' \, 11\,5/8'')^2} = \sqrt{139.624} = 11' \, 5\frac{5}{32}''$.

Chord $X\,5 = \sqrt{10^2 + (17' \, 2\,1/4'' - 13' \, 6\,1/32'')^2} = \sqrt{113.502} = 10' \, 7\frac{27}{32}''$.

Chord $X\,7 = \sqrt{10^2 + (19' \, 3\,11/16'' - 17' \, 2\,1/4'')^2} = \sqrt{104.494} = 10' \, 2\frac{21}{32}''$.

Chord $X\,9 = \sqrt{10^2 + (20' \, 0'' - 19' \, 3\,11/16'')^2} = \sqrt{100.4798} = 10' \, 0\frac{9}{32}''$.

The lengths of the chord panels just computed are those of straight members connecting the apexes of the upper chord, which is generally the best mode of constructing the curved chord. But if the panels of the upper chord are to be curved to the segmental arc, their panel lengths will be greater and are accordingly to be computed.

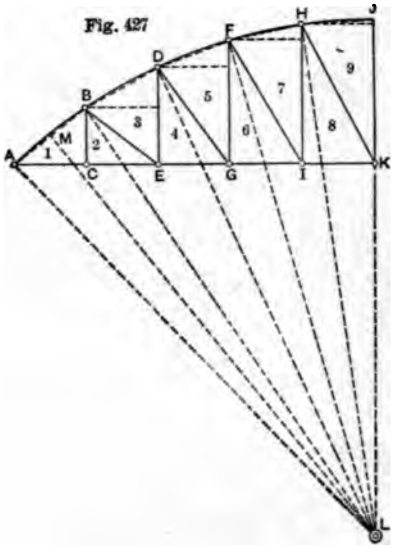
306. Length of Arc Panels.—Length of X 1 or arc panel $A\,B$ in Fig. 427.

Bisect chord panel $A\,B$ by perpendicular $M\,L$; draw radii $A\,L$ and $B\,L$, thus producing two equal right-angled triangles $A\,M\,L$, $B\,M\,L$. Then in the triangle $B\,M\,L$:

$BL : BM :: 1 : \sin BLM$. Or $72' \, 6\frac{1}{16}'' : \dfrac{12' \, 9\,7/16''}{2} :: 1 : \sin 5° \, 3' \, 31''$.

Then 2 (5° 3′ 31″) = 10° 7′ 2″ = angle at centre L subtended by arc $A\,B$. 180° = 648000″. 10° 7′ 2″ = 36422″. $\pi \times 72'\,6\frac{1}{16}'' = 227.78'$.

Fig. 427



Therefore 648000″ : 36422″ : 227.78′ : 12′ $9\frac{5}{8}''$ = length of arc panel $A\,B$.

Then 12′ $9\frac{5}{8}''$ − 12′ $9\frac{7}{16}''$ = $\frac{3}{16}''$ = excess of arc over chord.

Length of X 3 or arc panel $B\,D$ in Fig. 427.

Proceeding in like manner as before:

72′ $6\frac{1}{16}''$: $\frac{11'\,5\,5/32''}{2}$: : 1 : sin 4° 31′ 18″. And 9° 2′ 36″ = 32556″.

648000″ : 32556″ : : 227.78′ : 11′ $5\frac{5}{16}''$ = length of arc panel $B\,D$.

Then 11′ $5\frac{5}{16}''$ − 11′ $5\frac{5}{32}''$ = $\frac{5}{32}''$ = excess of arc over chord.

Length of X 5 or arc panel $D\,F$ in Fig. 427.

72′ $6\frac{1}{16}''$: $\frac{10'\,7\,27/32''}{2}$: : 1 : sin 4° 12′ 51″. And 8° 25′ 42″ = 30342″.

648000″ : 30342″ : 227.78′ : 10′ $7\frac{31}{32}''$ = length of arc panel $D\,F$.

Then $10'\ 7\frac{31}{32}'' - 10'\ 7\frac{27}{32}'' = \frac{1}{8}''$ = excess of arc over chord.

Length of X 7 or arc panel $F\ H$ in Fig. 427.

$72'\ 6\frac{1}{16}'' : \frac{10'\ 2\ 21/32''}{2} :: 1 : \sin 4°\ 2'\ 35''$. And $8°\ 5'\ 10'' = 29110''$.

$648000'' : 29110'' :: 227.78' : 10'\ 2\frac{25}{32}''$ = length of arc panel $F\ H$.

Then $10'\ 2\frac{25}{32}'' - 10'\ 2\frac{21}{32}'' = \frac{1}{8}''$ = excess of arc over chord.

Length of X 9 or arc panel $H\ J$ in Fig. 427.

$72'\ 6\frac{1}{16}'' : \frac{10'\ 0\ 9/32''}{2} : 1 :: \sin 3°\ 57'\ 52''$. And $7°\ 55'\ 44'' = 28544''$.

$648000'' : 28544'' :: 227.78' : 10'\ 0\frac{13}{32}''$ = length of arc panel $H\ J$.

Then $10'\ 0\frac{13}{32}'' - 10'\ 0\frac{9}{32}'' = \frac{1}{8}''$ = excess of arc over chord.

Example 15.—Same Truss with Reversed Diagonals (Fig. 428)

307. Lengths of Diagonals.—No lengths of any members are changed from those found in Example 14, excepting the lengths of the reversed diagonals.

Diagonal 2 3 $= \sqrt{10^2 + (13'\ 6\ 1/32'')^2} = \sqrt{282.3203} = 16'\ 9\frac{5}{8}''$.

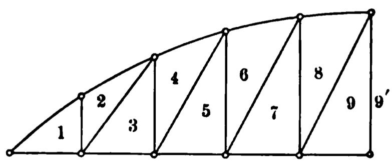


Fig. 428

Diagonal 4 5 $= \sqrt{10^2 + (17'\ 2\ 1/4'')^2} = \sqrt{395.4102} = 19'\ 10\frac{5}{8}''$.

Diagonal 6 7 $= \sqrt{10^2 + (19'\ 3\ 11/16'')^2} = \sqrt{472.7715} = 21'\ 8\frac{29}{32}''$.

Diagonal 8 9 $= \sqrt{10^2 + 20^2} = \sqrt{500} = 22'\ 4\frac{11}{32}''$.

Example 16.—Same Truss with Web Members in Howe Arrangement (Fig. 429)

$Y\,1 = 15'\ 0''$.

308. Computations.—Lengths of dotted verticals are same as verticals in Examples 14 and 15. Lengths of web members are alone changed.

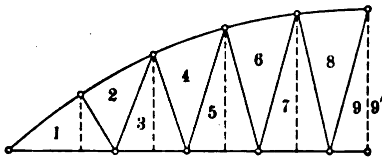


Fig. 429

Diagonal 1 2 = $\sqrt{5^2 + (7'\ 11\ 5/8'')^2} = 9'\ 4\frac{29}{32}''$.

Diagonals 2 3 and 3 4 = $\sqrt{5^2 + (13'\ 6\ 1/32'')^2} = 14'\ 4\frac{25}{32}''$.

Diagonals 4 5 and 5 6 = $\sqrt{5^2 + (17'\ 2\ 1/4'')^2} = 17'\ 10\frac{13}{16}''$.

Diagonals 6 7 and 7 8 = $\sqrt{5^2 + (18'\ 3\ 11/16'')^2} = 19'\ 11\frac{11}{32}''$.

Diagonal 8 9 = $\sqrt{5^2 + 20^2} = 20'\ 7\frac{11}{32}''$.

Example 17.—Truss with Segmental Chords and Equidistant Verticals (Fig. 430)

Nearly similar in form to Example 9 of Chapter IV.

309. Description.—Span, 100 ft.; rise of upper chord, 20 ft.; rise of lower chord, 5 ft., making the truss 15 ft. deep at middle; 10 panels.

The lengths of the chord and arc panels of the upper chord are the same as in Example 15, but those of the lower chord, verticals, and diagonals must be computed. Radius of upper chord = $72'\ 6\frac{1}{16}''$ as before. Heights of apexes A, B, D, F, H, and J above the horizontal span line $A\,N$ are the same as in Example 15.

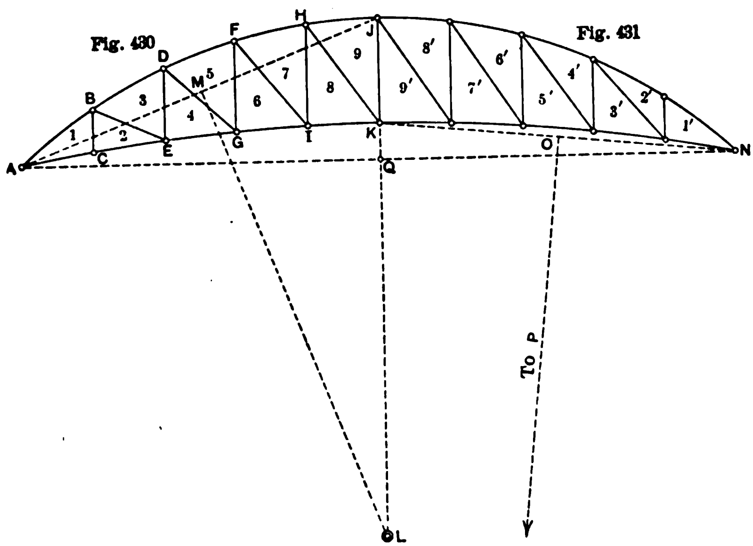
310. Radii of Segmental Chords.—Joining $K\,N$ and bisecting $K\,N$ by the perpendicular $O\,P$, this intersects the extended middle vertical at P, the centre of the segmental lower chord.

$$K\,N = \sqrt{50^2 + 5^2} = \sqrt{2525} = 50'\ 3''. \quad K\,O = \frac{50'\ 3''}{2} = 25'\ 1\frac{1}{2}''.$$

Since the two right-angled triangles $K\,P\,O$ and $K\,N\,Q$ have the common angle $N\,K\,P$, their similar sides are proportional.

Or $5' : 50'\ 3'' :: 25'\ 1\frac{1}{2}'' : 252'\ 6\frac{1}{8}''$ = radius of the lower chord.

Length of vertical ordinate through $A = Q\,P = 252'\ 6\frac{1}{8}'' - 5' = 247'\ 6\frac{1}{8}''$.



311. Heights of Apexes.—Heights of apexes of lower chord above span line $A\,N$.

By equation for circle with centre P at origin of coördinates, $y = \sqrt{r^2 - x^2}$.

Height of apex $C = \sqrt{(252'\ 6\,1/8'')^2 - 40^2} - 247'\ 6\frac{1}{8}'' = 1'\ 9\frac{3}{4}''$.

Height of apex $E = \sqrt{(252'\ 6\,1/8'')^2 - 30^2} - 247'\ 6\frac{1}{8}'' = 3'\ 2\frac{17}{32}''$.

Height of apex $G = \sqrt{(252'\,6\,1/8'')^2 - 20^2} - 247'\,6\frac{1}{8}'' = 4'\,2\frac{15}{32}''.$

Height of apex $I = \sqrt{(252'\,6\,1/8'')^2 - 10^2} - 247'\,6\frac{1}{8}'' = 4'\,9\frac{5}{8}''.$

Height of apex $K = 5'\,0''$ by construction $= 5'\,0''.$

312. Lengths of Web Members.—Length of any vertical = difference in heights of its ends.

Vertical 1 2 $= 7'\,11\frac{5}{8}'' - 1'\,9\frac{3}{4}'' = 6'\,1\frac{7}{8}''.$

Vertical 3 4 $= 13'\,6\frac{1}{32}'' - 3'\,2\frac{17}{32}'' = 10\,3\frac{1}{2}''.$

Vertical 5 6 $= 17'\,2\frac{1}{4}'' - 4'\,2\frac{15}{32}'' = 12'\,11\frac{25}{32}''.$

Vertical 7 8 $= 19'\,3\frac{11}{16}'' - 4'\,9\frac{5}{8}'' = 14'\,6\frac{1}{16}''.$

Vertical 9 9′ $= 20'\,0'' - 5'\,0'' = 15'\,0''.$

Length of any diagonal $= \sqrt{10^2 + (\text{difference in heights of ends})^2}.$

Diagonal 2 3 $= \sqrt{10^2 + (7'\,11\,5/8'' - 3'\,2\,17/32'')^2} = 11'\,0\frac{11}{32}''.$

Diagonal 4 5 $= \sqrt{10^2 + (13'\,6\,1/32'' - 4'\,2\,15/32'')^2} = 13'\,7\frac{27}{32}''.$

Diagonal 6 7 $= \sqrt{10^2 + (17'\,2\,1/4'' - 4'\,9\,5/8'')^2} = 15'\,11\frac{1}{32}''.$

Diagonal 8 9 $= \sqrt{10^2 + (19'\,3\,11/16'' - 5'\,0'')^2} = 17'\,5\frac{15}{32}''.$

313. Length of Chord Panels.—Length of any chord panel is found by the same formula.

Chord $Y\,1 = \sqrt{10^2 + (1'\,9\,3/4'')^2} = 10'\,1\frac{31}{32}''.$

Chord $Y\,2 = \sqrt{10^2 + (3'\,2\,17/32'' - 1'\,9\,3/4'')^2} = 10'\,1\frac{5}{32}''.$

Chord $Y\,4 = \sqrt{10^2 + (4'\,2\,15/32'' - 3'\,2\,17/32'')^2} = 10'\,0\frac{19}{32}''.$

Chord $Y\,6 = \sqrt{10^2 + (4'\,9\,5/8'' - 4'\,2\,15/32'')^2} = 10'\,0\frac{7}{32}''.$

Chord $Y\,8 = \sqrt{10^2 + (5'\,0'' - 4'\,9\,5/8'')^2} = 10'\,0\frac{1}{32}''.$

314. Lengths of Arc Panels.—Lengths of arc panels of the lower chord.

For arc panel $Y\,1$ or $A\,C$.

Join $A\,C$ and draw radii $A\,P$, $C\,P$; bisect $A\,C$ at M by perpendicular $M\,P$ drawn through centre P, as in Fig. 427 (not shown in Fig. 430). Then as in Example 17. Radius $L\,C$: half chord $Y\,1$: : 1 : sin angle $A\,P\,M$ at centre P. Or 252′ 6 1/8″ : 5′ 1″ : : 1 : sin 1° 9′ 13″. 2° 18′ 26″ = 8306″.

$$648000'' : 8306'' :: \pi\,(252'\,6\tfrac{1}{8}'') : 10'\,2''$$

And $10'\,2'' - 10'\,1\frac{31}{32}'' = 0\frac{1}{32}''$ = excess of arc over chord.

For arc panel $Y\,2$ or $C\,E$.

$252'\,6\frac{1}{8}'' : 5'\,0\frac{19}{32}'' :: 1 : \sin 1°\,8'\,45''$. $2°\,17'\,30'' = 8250''$.

$648000'' : 8250'' :: \pi\,(252'\,6\frac{1}{8}'') : 10'\,1\frac{3}{16}''$ = arc $C\,E$.

$10'\,1\frac{3}{16}'' - 10'\,1\frac{5}{32}'' = 0\frac{1}{32}''$ = excess of arc.

For arc panel $Y\,4$ or $E\,G$.

$252'\,6\frac{1}{8}'' : 5'\,0\frac{5}{16}'' :: 1 : \sin 1°\,8'\,26''$. $2°\,16'\,52'' = 8212''$.

$648000'' : 8212 :: \pi\,(252'\,6\frac{1}{8}'') : 10'\,0\frac{5}{8}''$ = arc $E\,G$.

$10'\,0\frac{5}{8}'' - 10'\,0\frac{19}{32}'' = \frac{1}{32}''$ = excess of arc.

For arc panel $Y\,6$ or $G\,I$.

$252'\,6\frac{1}{8}'' : 5'\,0\frac{1}{8}'' :: 1 : \sin 1°\,8'\,13''$. $2°\,16'\,26'' = 8186''$.

$648000'' : 8186'' :: \pi\,(252'\,6\frac{1}{8}'') : 10'\,0\frac{1}{4}''$ = arc $G\,I$.

$10'\,0\frac{1}{4}'' - 10'\,0\frac{7}{32}'' = \frac{1}{32}''$ = excess of arc.

For arc panel $Y\,8$ or $I\,K$.

$252'\,6\frac{1}{8}'' : 5'\,0\frac{1}{32}'' :: 1 : \sin 1°\,8'\,7''$. $2°\,16'\,14'' = 8174''$.

$$648000'' : 8174'' :: \pi\,(252'\,6\frac{1}{8}'') : 10'\,0\frac{1}{16}'' = \text{arc } I\,K.$$

$$10'\,0\frac{1}{16}'' - 10'\,0\frac{1}{32}'' = 0\frac{1}{32}'' = \text{excess of arc.}$$

EXAMPLE 18.—SAME TRUSS WITH REVERSED DIAGONALS (Fig. 431)

315. Lengths of Diagonals.—Lengths of diagonals are alone changed.

$$\text{Diagonal } 2\,3 = \sqrt{10^2 + (13'\,6\,1/32'' - 1'\,9\,3/4'')^2} = 15'\;4\frac{19}{32}''.$$

$$\text{Diagonal } 4\,5 = \sqrt{10^2 + (17'\,2\,1/4'' - 3'\,2\,17/32'')^2} = 17'\;2\frac{7}{32}''.$$

$$\text{Diagonal } 6\,7 = \sqrt{10^2 + (19'\,3\,11/16'' - 4'\,2\,15/32'')^2} = 18'\;1\frac{11}{32}''.$$

$$\text{Diagonal } 8\,9 = \sqrt{10^2 + (20'\,0'' - 4'\,9\,5/8'')^2} = 18'\;2\frac{5}{16}''.$$

EXAMPLE 19.—SEMICIRCULAR CRESCENT TRUSS (Fig. 432)

316. Variations from Example 10 of Chapter IV.—Span, 100 ft.; rise of upper chord, 50 ft.; rise of lower chord, 37.5 ft., making depth of truss at middle 12.5 ft. Upper and lower chords are each divided into 14 equal arc panels by radials as shown. If these radials all radiate from the centre P, the calculations become far more difficult and the appearance of the truss is less satisfactory.

317. Radii of Chords.—Radius of upper chord = 50 ft. For obtaining radius of lower chord, in Fig. 433 join apexes O and Q, and bisect $O\,Q$ by perpendicular $R\,S$, intersecting extended middle vertical at S, the centre of the segmental lower chord.

Since the two right-angled triangles $R\,O\,S$ and $O\,P\,Q$ have the common angle $R\,O\,P$, their corresponding sides must be proportional to each other.

Hence $O\,P : O\,R :: O\,Q : O\,S$ = required radius of lower chord.

$O\,Q = \sqrt{(37'\,6'')^2 + 50^2} = 62'\,6''$. $O\,R = 31'\,3''$.

Then $37'\,6'' : 31'\,3'' :: 62'\,6'' : 52'\,1''$ = radius of lower chord.

318. Coördinates of Upper Apexes.—Coördinates of apexes of upper chord, origin at P.

Angle at centre subtended by one panel of upper chord = $\frac{90°}{7} = 12°\,51'\,25\frac{5}{7}''$.

For apex A. $x = 50' \, 0''$. $y = 0' \, 0''$.

For apex B. Join $B\,P$ in Fig. 432; drop vertical from B to $A\,P$.

$$x = 50' \cos 12^\circ \, 51' \, 25\frac{5}{7}'' = 48' \, 8\frac{31}{32}''.$$

Fig. 432

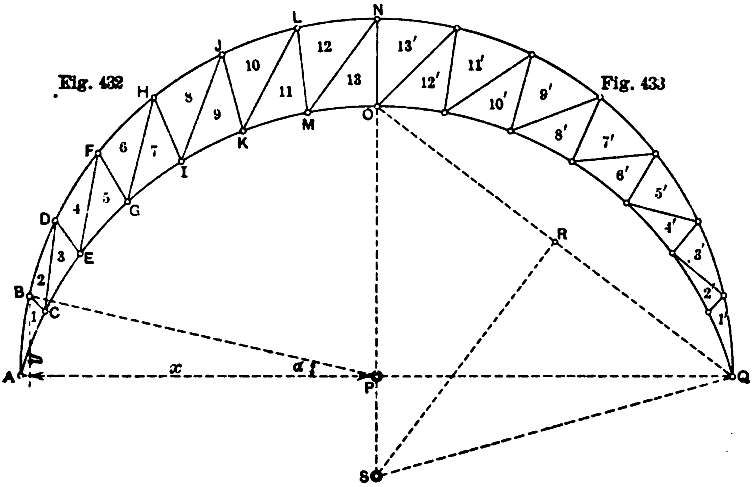


Fig. 433

$$y = 50' \sin 12^\circ \, 51' \, 25\frac{5}{7}'' = 11' \, 1\frac{1}{2}''.$$

For apex D.

$$x = 50' \cos 2 \, (12^\circ \, 51' \, 25\frac{5}{7}'') = 45' \, 0\frac{9}{16}''.$$

$$y = 50' \sin 2 \, (12^\circ \, 51' \, 25\frac{5}{7}'') = 21' \, 8\frac{11}{32}''.$$

For apex F.

$$x = 50' \cos 3 \, (12^\circ \, 51' \, 25\frac{5}{7}'') = 39' \, 1\frac{3}{32}''.$$

$$y = 50' \sin 3 \, (12^\circ \, 51' \, 25\frac{5}{7}'') = 31' \, 2\frac{3}{32}''.$$

For apex H.

$$x = 50' \cos 4 \, (12^\circ \, 51' \, 25\frac{5}{7}'') = 31' \, 2\frac{3}{32}''.$$

$$y = 50' \sin 4 \, (12^\circ \, 51' \, 25\frac{5}{7}'') = 39' \, 1\frac{3}{32}''.$$

For apex J.

$$x = 50' \cos 5\ (12^\circ\ 51'\ 25\frac{5}{7}'') = 21'\ 8\frac{11}{32}''.$$

$$y = 50' \sin 5\ (12^\circ\ 51'\ 25\frac{5}{7}'') = 45'\ 0\frac{9}{16}''.$$

For apex L.

$$x = 50' \cos 6\ (12^\circ\ 51'\ 25\frac{5}{7}'') = 11'\ 1\frac{1}{2}''.$$

$$y = 50' \sin 6\ (12^\circ\ 51'\ 25\frac{5}{7}'') = 48'\ 8\frac{31}{32}''.$$

For apex N. $x = 0'\ 0''$. $y = 50'\ 0''$.

319. Angle at Centre Subtended by Panel of Lower Chord.—In the right-angled triangle $O\,S\,R$, $O\,S : O\,R :: 1 : \sin O\,S\,R$.

Or $52'\ 1'' : 31'\ 3'' :: 1 : \sin 36^\circ\ 52'\ 11\frac{1}{2}''$.

Angle $O\,S\,Q$ at centre S subtended by one-half the lower chord $= 2\ (36^\circ\ 52'\ 11\frac{1}{2}'') = 73^\circ\ 44'\ 23''$.

Hence $\dfrac{73^\circ\ 44'\ 23''}{7} = 10^\circ\ 32'\ 3\frac{2}{7}''$ = angle at centre S subtended by one panel of the lower chord.

Join $A\,S = 52'\ 1''$. $P\,S = 52'\ 1'' - 37'\ 6'' = 14'\ 7''$.

The angle $P\,A\,S$ = angle between $A\,S$ and a horizontal drawn through S.

Then $52'\ 1'' : 14'\ 7'' :: 1 : \sin 16^\circ\ 15'\ 37''$ = angle $P\,A\,S$.

320. Coördinates of Lower Apexes.—Coördinates of apexes of lower chord referred to origin at P.

For apex A. $x = 50'\ 0''$. $y = 0'\ 0''$.

For apex C.

$$x = 52'\ 1'' \cos\ [16^\circ\ 15'\ 37'' + 10^\circ\ 32'\ 3\frac{2}{7}''] = 46'\ 6''.$$

$$y = 52'\ 1'' \sin\ [16^\circ\ 15'\ 37'' + 10^\circ\ 32'\ 3\frac{2}{7}''] - 14'\ 7'' = 8'\ 10\frac{17}{32}''.$$

For apex E.

$$x = 52'\ 1'' \cos\ [16^\circ\ 15'\ 37'' + 2\ (10^\circ\ 32'\ 3\frac{2}{7}'')] = 41'\ 4\frac{31}{32}''.$$

$$y = 52'\ 1'' \sin\ [16^\circ\ 15'\ 37'' + 2\ (10^\circ\ 32'\ 3\frac{2}{7}'')] - 14'\ 7'' = 17'\ 0''.$$

For apex G.

$$x = 52'\ 1''\cos\ [16°\ 15'\ 37'' + 3\ (10°\ 32'\ 3\tfrac{2}{7}'')] = 34'\ 11\tfrac{17}{32}''.$$

$$y = 52'\ 1''\ \sin\ [16°\ 15'\ 37'' + 3\ (10°\ 32'\ 3\tfrac{2}{7}'')] - 14'\ 7'' = 24'\ 4\tfrac{15}{32}''.$$

For apex I.

$$x = 52'\ 1''\cos\ [16°\ 15'\ 37'' + 4\ (10°\ 32'\ 3\tfrac{2}{7}'')] = 27'\ 3\tfrac{1}{2}''.$$

$$y = 52'\ 1''\ \sin\ [16°\ 15'\ 37'' + 4\ (10°\ 32'\ 3\tfrac{2}{7}'')] - 14'\ 7'' = 29'\ 9\tfrac{5}{16}''.$$

For apex K.

$$x = 52'\ 1''\cos\ [16°\ 15'\ 37'' + 5\ (10°\ 32'\ 3\tfrac{2}{7}'')] = 18'\ 8\tfrac{11}{16}''.$$

$$y = 52'\ 1''\ \sin\ [16°\ 15'\ 37'' + 5\ (10°\ 32'\ 3\tfrac{2}{7}'')] - 14'\ 7'' = 34'\ 0\tfrac{7}{32}''.$$

For apex M.

$$x = 52'\ 1''\cos\ [16°\ 15'\ 37'' + 6\ (10°\ 32'\ 3\tfrac{2}{7}'')] = 9'\ 6\tfrac{1}{4}''.$$

$$y = 52'\ 1''\ \sin\ [16°\ 15'\ 37'' + 6\ (10°\ 32'\ 3\tfrac{2}{7}'')] - 14'\ 7'' = 36'\ 7\tfrac{1}{2}''.$$

For apex O. $x = 0'\ 0''$. $y = 37'\ 6''$.

321. Table of Apex Coördinates.—For greater convenience in the succeeding computations, the coördinates of the apexes of the upper and lower chord are tabulated as follows:

Upper chord			Lower chord		
Apex	*X*.	*Y*	Apex	*X*.	*Y*
A.	50′ 0″	0′ 0″	*A*.	50′ 0″	0′ 0″
B.	48′ 8 31/32″	11′ 1 1/2″	*C*.	46′ 6″	8′ 10 17/32″
D.	45′ 0 9/16″	21′ 8 11/32″	*E*.	41′ 4 31/32″	17′ 0″
F.	39′ 1 3/32″	31′ 2 3/32″	*G*.	34′ 11 15/32″	24′ 4 15/32″
H.	31′ 2 3/32″	39′ 1 3/32″	*I*.	27′ 3 1/2″	29′ 9 5/16″
J.	21′ 8 11/32″	45′ 0 9/16″	*K*.	18′ 8 11/16″	34′ 0 7/32″
L.	11′ 1 1/2″	48′ 8 31/32″	*M*.	9′ 6 1/4″	36′ 7 1/2″
N.	0′ 0″	50′ 0″	*O*.	0′ 0″	50′ 0″

322. Lengths of Radials Between Apexes.—Difference of abscissas x of ends of radial = its horizontal projection.

Differences of ordinates y of ends of radial = its vertical projection.

Length of radial = $\sqrt{(\text{horizontal projection})^2+(\text{vertical projection})^2}$.

Radial 1 2 = $\sqrt{(2'\,2\,31/32'')^2 + (2'\,2\,31'')^2} = 3'\,0\frac{1}{8}''$.

Radial 3 4 = $\sqrt{(3'\,7\,19/32'')^2 + (4'\,8\,11/32'')^2} = 5'\,11\frac{1}{4}''$.

Radial 5 6 = $\sqrt{(4'\,1\,9/16'')^2 + (6'\,9\,5/8'')^2} = 7'\,11\frac{1}{2}''$.

Radial 7 8 = $\sqrt{(3'\,10\,16/32'')^2 + (9'\,3\,25/32'')^2} = 10'\,1\frac{1}{16}''$.

Radial 9 10 = $\sqrt{(2'\,11\,21/32'')^2 + (11'\,0\,11/32'')^2} = 11'\,5\frac{1}{16}''$.

Radial 11 12 = $\sqrt{(1'\,7\,1/4'')^2 + (12'\,1\,15/32'')^2} = 12'\,2\frac{3}{4}''$.

Radial 13 13′ = 12′ 6″ by construction.

323. Lengths of Diagonals.—

Length of diagonal =

$$\sqrt{(\text{horizontal projection})^2 + (\text{vertical projection})^2}$$

Diagonal 2 3 = $\sqrt{(1'\,5\,7/16'')^2 + (12'\,9\,13/16)^2} = 12'\,10\frac{13}{16}''$.

Diagonal 4 5 = $\sqrt{(2'\,3\,7/8'')^2 + (14'\,2\,31/32'')^2} = 14'\,4\frac{3}{8}''$.

Diagonal 6 7 = $\sqrt{(3'\,9\,7/16'')^2 + (14'\,8\,5/8'')^2} = 15'\,2\frac{3}{8}''$.

Diagonal 8 9 = $\sqrt{(5'\,7\,5/32'')^2 + (15'\,3\,1/4'')^2} = 16'\,3\frac{5}{32}''$.

Diagonal 10 11 = $\sqrt{(7'\,7\,3/16'')^2 + (14'\,8\,3/4'')^2} = 16'\,6\frac{5}{32}''$.

Diagonal 12 13 = $\sqrt{(9'\,6\,1/4'')^2 + (13'\,4\,1/2'')^2} = 16'\,5''$.

324. Lengths of Members of Chords.—Since each chord is here divided into equal panels, it will only be necessary to compute the chord and arc lengths for one panel of each chord.

For upper chord.

$12°\,51'\,25\frac{5}{7}''$ = angle at centre P subtended by one panel, as before.

$2 \times 50' \sin\frac{1}{2}\,12°\,51'\,25\frac{5}{7}'' = 11'\,2\frac{11}{32}''$ = length of chord panel.

$180° = 648000''$. $12°\,51'\,25\frac{5}{7}'' = 46286''$.

Then $648000'' : 46286'' :: \pi \times 50' : 11'\, 2\frac{31}{32}'' =$ arc length of panel.

And $11'\, 2\frac{31}{32}'' - 11'\, 2\frac{11}{32}'' = \frac{5}{8}'' =$ excess of arc length over chord length.

For lower chord.

$10°\, 32'\, 3\frac{1}{7}'' =$ angle at centre S subtended by one panel.

$2 \times (52'\ 1'') \sin\frac{1}{2}\, 10°\, 32'\, 3\,\frac{1}{7}'' = 9'\, 6\,\frac{3}{4}'' =$ chord panel length.

$10°\, 32'\, 3\,\frac{1}{7}'' = 37923''$.

$648000'' : 37923'' :: \pi\,(52'\ 1'') : 9'\, 6\,\frac{29}{32}'' =$ arc panel length.

Then $9'\, 6\,\frac{29}{32}'' - 9'\, 6\,\frac{3}{4}'' = \frac{5}{32}'' =$ excess of arc length over chord length.

Example 20.—Same Truss with Reversed Diagonals (Fig. 433)

Since the coördinates of the apexes are the same as in the last example, the horizontal and vertical projections of the diagonals are easily computed by means of the Table of coördinates of apexes. Lengths of chord and radial members are unchanged.

325. Lengths of Diagonals.—

Diagonal 2 3 $= \sqrt{(7'\,4'')^2 + (5'\,10\,1/2'')^2} = 9'\,4\frac{3}{4}''$.

Diagonal 4 5 $= \sqrt{(10'\,1\,3/32'')^2 + (2'\,8\,1/8'')^2} = 10'\,5\,\frac{9}{32}''$.

Diagonal 6 7 $= \sqrt{(11'\,9\,19/32'')^2 + (1'\,4\,25/32'')^2} = 11'\,10\,\frac{19}{32}''$.

Diagonal 8 9 $= \sqrt{(12'\,5\,13/32'')^2 + (5'\,0\,7/8'')^2} = 13'\,5\,\frac{11}{32}''$.

Diagonal 10 11 $= \sqrt{(12'\,2\,3/32'')^2 + (8'\,5\,1/16'')^2} = 14'\,9\,\frac{21}{32}''$.

Diagonal 12 13 $= \sqrt{(11'\,1\,1/2'')^2 + (11'\,2\,31/32'')^2} = 15'\,9\,\frac{27}{32}''$.

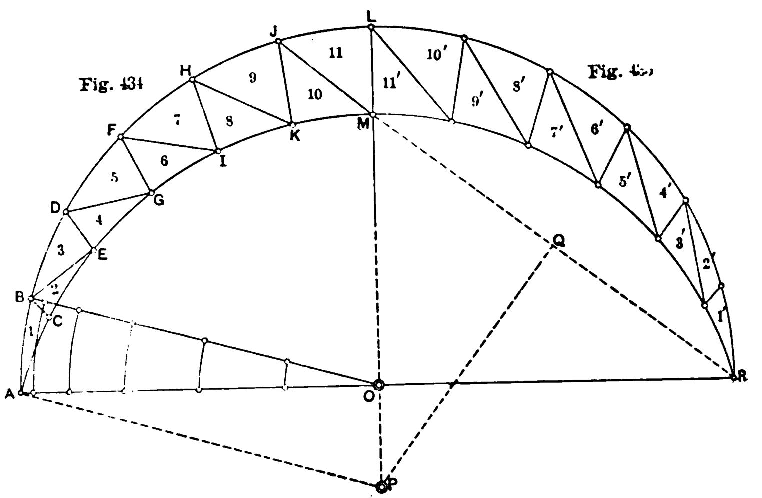
326. Crescent Truss with Radials from Centre of Upper Chord.—For a crescent truss with radials from centre of upper chord, the calculation of the lengths of the members is far more difficult than

for Example 19, since the lengths of the panels of lower chord are unequal. This truss with equal lengths of the upper and of the lower chord is more easily constructed and has a better appearance when built. Therefore it is recommended for practical use.

Example 21.—A Hemispherical Dome Supported by Crescent Trusses (Fig. 434)

327. Variations from Example 11 of Chapter IV.—Dome is 100 ft. in diameter and is supported by 12 complete crescent trusses intersecting in the middle vertical. Depth of truss at vertex = 12.5 ft. Each truss is divided into 12 equal panels in each chord, as in Fig. 434. Calculations of the lengths of chord and web members are made in the manner employed for Example 19. Since the span and rise of the chords of the crescent truss are the same as in that example, but the number of panels in each quadrant is 6 instead of 7, it becomes necessary to make complete calculations for this Example.

328. Radius of Lower Chord.—In Fig. 435, join $M\,R$ and bisect same by perpendicular $Q\,P$, cutting extended middle vertical at P, the centre of the lower chord.



$$M\,R = \sqrt{50^2 + (37'\,6'')^2} = 62'\,6''. \quad M\,Q = \frac{62'\,6''}{2}\;31'\,3''.$$

Then $37'\,6'' : 31'\,3'' :: 62'\,6'' : 52'\,1'' = M\,P =$ radius of lower chord.
And $52'\,1'' - 37'\,6'' = 14'\,7'' = O\,P$.

329. Angles at Centres Subtended by Chord Panels.—

$$\text{Tan } O\,A\,P = \frac{14'\,7''}{50'} = \tan 16°\,15'\,37''.$$

Also $\dfrac{90° - 16°\,15'\,37''}{6} = 12°\,17'\,23\dfrac{5}{6}''$ = angle at centre P subtended by one panel of lower chord.

And $\dfrac{90°}{6} = 15°$ = angle at centre O subtended by one panel of upper chord.

330. Coördinates of Upper Apexes.—Coördinates of apexes of upper chord referred to origin at O.

For apex A. $x = 50'\,0''$. $y = 0'\,0''$.

For apex B.

$$x = 50' \cos 15° = 48'\,3\frac{9}{16}''.$$

$$y = 50' \sin 15° = 12'\,11\frac{9}{32}''.$$

For apex D.

$$x = 50' \cos 2\,(15°) = 43'\,3\frac{5}{8}''.$$

$$y = 50' \sin 2\,(15°) = 25'\,0''.$$

For apex F.

$$x = 50' \cos 3\,(15°) = 35'\,4\frac{9}{32}''.$$

$$y = 50' \sin 3\,(15°) = 35'\,4\frac{9}{32}''.$$

For apex H.

$$x = 50' \cos 4\,(15°) = 25'\,0''.$$

$$y = 50' \sin 4\,(15°) = 43'\,3\frac{5}{8}''.$$

For apex J.

$$x = 50' \cos 5\,(15°) = 12'\,11\frac{9}{32}''.$$

$$y = 50' \sin 5\,(15°) = 48'\,3\frac{9}{16}''.$$

For apex L. $x = 0'\,0''$. $y = 50'\,0''$.

331. Coördinates of Lower Apexes.—Coördinates of apexes of lower chord referred to origin at O.

For apex A. $x = 50'\,0''$. $y = 0'\,0''$.

For apex C.

$$x = 52'\,1'' \cos\left(16°\,15'\,37'' + 12°\,17'\,23\frac{5}{6}''\right) = 45'\,9''.$$

$$y = 52'\,1'' \sin\left(16°\,15'\,37'' + 12°\,17'\,23\frac{5}{6}''\right) - 14'\,7'' = 10'\,3\frac{23}{32}''.$$

For apex E.

$$x = 52'\,1'' \cos\left[16°\,15'\,37'' + 2\left(12°\,17'\,23\frac{5}{6}''\right)\right] = 39'\,4\frac{27}{32}''.$$

$$y = 52'\,1'' \sin\left[16°\,15'\,37'' + 2\left(12°\,17'\,23\frac{5}{6}''\right)\right] - 14'\,7'' = 19'\,5\frac{23}{32}''.$$

For apex G.

$$x = 52'\,1'' \cos\left[16°\,15'\,37'' + 3\left(12°\,17'\,23\frac{5}{6}''\right)\right] = 31'\,3''.$$

$$y = 52'\,1'' \sin\left[16°\,15'\,37'' + 3\left(12°\,17'\,23\frac{5}{6}''\right)\right] - 14'\,7'' = 27'\,1''.$$

For apex I.

$$x = 52'\,1'' \cos\left[16°\,15'\,37'' + 4\left(12°\,17'\,23\frac{5}{6}''\right)\right] = 21'\,7\frac{31}{32}''.$$

$$y = 52'\,1'' \sin\left[16°\,15'\,37'' + 4\left(12°\,17'\,23\frac{5}{6}''\right)\right] - 14'\,7'' = 32'\,9\frac{3}{8}''.$$

For apex K.

$$x = 52'\,1'' \cos\left[16°\,15'\,37'' + 5\left(12°\,17'\,23\frac{5}{6}''\right)\right] = 11'\,1\frac{1}{32}''.$$

$$y = 52'\,1'' \sin\left[16°\,15'\,37'' + 5\left(12°\,17'\,23\frac{5}{6}''\right)\right] - 14'\,7'' = 36'\,3\frac{21}{32}''.$$

For apex M. $x = 0'\,0''$. $y = 37'\,6''$.

332. Table of Coördinates.—These coördinates are also tabulated.

Apex	x	y	Apex	x	y
A.	50′ 0″	0′ 0″	A.	50′ 0″	0′ 0″
B.	48′ 3 9/16″	12′ 11 9/32″	C.	45′ 9″	10′ 3 25/32″
D.	43′ 3 5/8″	25′ 0″	E.	39′ 4 27/32″	19′ 5 23/32″
F.	35′ 4 9/32″	35′ 4 9/32″	G.	31′ 3″	27′ 1″
H.	25′ 0″	43′ 3 5/8″	I.	21′ 7 31/32″	32′ 9 3/8″
J.	12′ 11 9/32″	48′ 3 9/16″	K.	11′ 1 1/32″	36′ 3 21/32″
L.	0′ 0″	50′ 0″	M.	0′ 0″	37′ 6″

333. Lengths of Radials and Diagonals.—Radials and diagonals are computed in the same manner as in Example 19.

Radial 1 2 = $\sqrt{(2'\,6\,9/16'')^2 + (2'\,7\,9/16'')^2} = 3'\,7\frac{15}{16}''.$

Radial 3 4 = $\sqrt{(3'\,10\,25/32'')^2 + (5'\,6\,9/32'')^2} = 6'\,9\frac{1}{8}''.$

Radial 5 6 = $\sqrt{(4'\,1\,9/32'')^2 + (8'\,3\,9/32'')^2} = 9'\,2\frac{27}{32}''.$

Radial 7 8 = $\sqrt{(3'\,4\,1/32'')^2 + (10'\,6\,1/4'')^2} = 11'\,0\frac{7}{16}''.$

Radial 9 10 = $\sqrt{(1'\,10\,1/4'')^2 + (11'\,11\,29/32'')^2} = 12'\,1\frac{5}{8}''.$

Radial 11 11′ = 12′ 6″ by construction.

Diagonal 2 3 = $\sqrt{(8'\,10\,23/32'')^2 + (6'\,6\,7/16'')^2} = 11'\,0\frac{7}{16}''.$

Diagonal 4 5 = $\sqrt{(12'\,0\,5/8'')^2 + (2'\,1'')^2} = 12'\,2\frac{25}{32}''.$

Diagonal 6 7 = $\sqrt{(13'\,8\,5/16'')^2 + (2'\,6\,29/32'')^2} = 12'\,11\frac{13}{32}''.$

Diagonal 8 9 = $\sqrt{(13'\,10\,31/32'')^2 + (6'\,11\,31/32'')^2} = 15'\,6\frac{29}{32}''.$

Diagonal 10 11 = $\sqrt{(12'\,11\,9/32'')^2 + (10'\,9\,9/16'')^2} = 16'\,10\frac{7}{32}''.$

334. Lengths of Chord and Arc Panels.—Lengths for members of upper chord.

Chord panel = $2 \times 50' \sin \frac{1}{2}\, 15° = 13'\,0\frac{5}{8}''.$

$$15° = 54000''.$$

$648000'' : 54000'' : : 50' \times \pi : 13'\,1\frac{1}{16}''$ = length of arc panel.

Then $13'\,1\frac{1}{16}'' - 13'\,0\frac{5}{8}'' = \frac{7}{16}''$ = excess of arc over chord.

Lengths of members of lower chord.

Chord panel = $2 \times 52'\,1'' \sin \frac{1}{2}\, 12°\,17'\,23\frac{5}{6}'' = 11'\,1\frac{13}{16}''.$

$12°\,17'\,23\frac{5}{6}'' = 44244.''$

$648000'' : 44244'' :: \pi\ (52''\ 1'') : 11'\ 2\frac{1}{16}'' =$ length of arc panel.

Then $11'\ 2\frac{1}{16}'' - 11'\ 1\frac{13}{16}'' = \frac{1}{4}'' =$ excess of arc over chord.

EXAMPLE 22.—SAME CRESCENT TRUSS WITH REVERSED DIAGONALS (Fig. 435)

335. Lengths of Diagonals.—Lengths of diagonals are alone changed from those in Example 21.

Diagonal 2 3 $= \sqrt{(2'\ 5\ 3/8'')^2 + (14'\ 8\ 9/32'')^2} = 14'\ 10\frac{23}{32}''$.

Diagonal 4 5 $= \sqrt{(4'\ 0\ 9/16'')^2 + (15'\ 10\ 9/16'')^2} = 16'\ 4\frac{21}{32}''$.

Diagonal 6 7 $= \sqrt{(6'\ 3'')^2 + (16'\ 2\ 5/8'')^2} = 17'\ 4\frac{9}{16}''$.

Diagonal 8 9 $= \sqrt{(8'\ 8\ 11/16'')^2 + (15'\ 6\ 3/16'')^2} = 17'\ 9\frac{19}{32}''$.

Diagonal 10 11 $= \sqrt{(11'\ 1\ 1/32'')^2 + (13'\ 8\ 11/32'')^2} = 17'\ 7\frac{7}{16}''$.

A. Lengths of purlins with axes curved in horizontal planes.

336. Radii of Purlin Axes.—The axes and arc of each purlin lie in a horizontal plane circle drawn through the corresponding apex of the upper chord.

The 50′ cos angle between radius drawn to the apex and a horizontal = radius of this horizontal circle.

$50'\cos 0° = 50'\ 0'' =$ radius of horizontal plate or purlin at *A*.

$50'\cos 15° = 48'\ 3\frac{9}{16}'' =$ radius of axis of purlin at *B*.

$50'\cos 2\ (15°) = 43'\ 3\frac{5}{8}'' =$ radius of axis of purlin at *D*.

$50'\cos 3\ (15°) = 35'\ 4\frac{9}{32}'' =$ radius of axis of purlin at *F*.

$50'\cos 4\ (15°) = 25'\ 0'' =$ radius of axis of purlin at *H*.

$50'\cos 5\ (15°) = 12'\ 11\frac{9}{32}'' =$ radius of axis of purlin at *J*.

$50'\cos 6\ (15) = 0'\ 0'' =$ radius of axis of purlin at vertex of dome.

337. Lengths of Purlin Axis. Chords.—Each purlin subtends an angle of 15° at the centre of the horizontal circle containing it.

Then 2 radius $\sin \frac{1}{2} 15° =$ length of chord of purlin arc.

$2 \times 50' \sin 7° 30' = 13' 0 \frac{5}{8}''$ for purlin at A.

$2 \times 48' 3 \frac{9}{16}'' \sin 7° 30' = 12' 7 \frac{5}{16}''$ for purlin at B.

$2 \times 43' 3 \frac{5}{8}'' \sin 7° 30' = 11' 3 \frac{21}{32}''$ for purlin at D.

$2 \times 35' 4 \frac{9}{32}'' \sin 7° 30' = 9' 2 \frac{3}{4}''$ for purlin at F.

$2 \times 25' 0'' \sin 7° 30' = 6' 6 \frac{5}{16}''$ for purlin at H.

$2 \times 12' 11 \frac{9}{32}'' \sin 7° 30' = 3' 4 \frac{17}{32}''$ for purlin at J.

338. Purlin Axis Vertical or Horizontal.—If in this case the principal axis of cross-section of the purlin be vertical or horizontal, it will be necessary to insert iron or steel wedge fillers between the purlin and the upper chord as well as between it and the curved rafters. The purlin is then bent sidewise only to the required curve, which makes the simplest mode of construction.

339. Purlin Axis Radial.—Or the main axis of its cross-section may be perpendicular to the upper chord, when no fillers will be required between chord and purlin or under the curved rafters, but the purlin must then be bent edgewise as well as sidewise to bring its axis into the desired curve. The arc length of the axis is the same in both cases. $15° = 54000''$.

$648000'' : 54000'' :: \pi \times$ radius : arc length of purlin.

$648000'' : 54000'' :: \pi \times 50' 0'' : 13' 1 \frac{1}{16}'' =$ arc purlin at A.

$648000'' : 54000'' :: \pi \times 48' 3 \frac{9}{16}'' : 12' 7 \frac{23}{32}'' =$ arc purlin at B.

$648000'' : 54000'' :: \pi \times 43' 3 \frac{5}{8}'' : 11' 4 \frac{1}{32}'' =$ arc purlin at D.

$648000'' : 54000'' :: \pi \times 35' 4 \frac{9}{32}'' : 9' 3 \frac{11}{16}'' =$ arc purlin at F.

$648000'' : 54000'' :: \pi \times 25' 0'' : 6' 6 \frac{17}{32}'' =$ arc purlin at H.

$$648000'' : 54000'' :: \pi \times 12'\,11\frac{9}{32}'' : 3'\,4\frac{21}{32}'' = \text{arc purlin at } J.$$

And $13'\,1\frac{1}{16}'' - 13'\,0\frac{5}{8}'' = \frac{7}{16}'' =$ excess of arc over chord at A.

$12'\,7\frac{23}{32}'' - 12'\,7\frac{5}{16}'' = \frac{13}{32}'' =$ excess of arc over chord at B.

$11'\,4\frac{1}{32}'' - 11'\,3\frac{21}{32}'' = \frac{3}{8}'' =$ excess of arc over chord at D.

$9'\,3\frac{1}{16}'' - 9'\,2\frac{3}{4}'' = \frac{5}{16}'' =$ excess of arc over chord at F.

$6'\,6\frac{17}{32}'' - 6'\,6\frac{5}{16}'' = \frac{7}{32}'' =$ excess of arc over chord at H.

$3'\,4\frac{21}{32}'' - 3'\,4\frac{17}{32}'' = \frac{1}{8}'' =$ excess of arc over chord at J.

B. Lengths of purlins with axes curved in great circles.

340. Description.—Main axis of cross-section of purlin perpendicular to upper chord and curved rafters. Hence the purlin is bent sidewise and edgewise. No fillers. If the interior of the dome be visible from below, its appearance will not be as good as in case *A*, since the successive purlins in a ring do not form a continuous curve between the apexes, and the divergence increases toward the vertex. But all purlins have the same curvature and there are other structural advantages. The curved axes of the purlins have a uniform radius, here = 50′, but the angles subtended at the centres of the great circles vary. Chord lengths of purlin axes are the same as in case *A*.

341. Arc Lengths of Purlin Axes.—

Then $\sin\frac{1}{2}\text{ angle at centre} = \frac{\text{purlin chord}}{2 \times 50'} = \frac{\text{chord}}{100'} = \frac{1}{2}$ angle at centre.

$\frac{13'\,0\,5/8''}{100'} = \sin 7°\,30'$. Angle for apex $A = 15° = 54000''$.

$\frac{12'\,7\,5/16''}{100'} = \sin 7°\,14'\,38''$. Angle for apex $B = 14°\,29'\,16'' = 52156''$

$\frac{11'\,3\,21/32''}{100'} = \sin 6°\,29'\,28''$. Angle for apex $D = 12°\,58'\,58'' = 46736''$.

$\frac{9'\,2\,3/4''}{100'} = \sin 5°\,17'\,44''$. Angle for apex $F = 10°\,35'\,28'' = 38128''$.

$$\frac{6'\ 6\ 5/16''}{100'} = \sin 3°\ 44'\ 30''. \quad \text{Angle for apex } H = 7°\ 29'\ 0'' = 26940''.$$

$$\frac{3'\ 4\ 19/32''}{100'} = \sin 1°\ 56'\ 8''. \quad \text{Angle for apex } J = 3'\ 52'\ 16'' = 13936''.$$

Then 648000'' : angle at centre : : $\pi \times R$: arc length.

$$648000'' : 54000'' : : 157.08' : 13'\ 1\frac{1}{16}'' = \text{arc purlin at } A.$$

$$648000'' : 52156'' : : 157.08' : 12'\ 7\frac{23}{32}'' = \text{arc purlin at } B.$$

$$648000'' : 46736'' : : 157.08' : 11'\ 3\frac{15}{16}'' = \text{arc purlin at } D.$$

$$648000'' : 38128'' : : 157.08' : 9'\ 2\frac{29}{32}'' = \text{arc purlin at } F.$$

$$648000'' : 26940'' : : 157.08' : 6'\ 6\frac{3}{8}'' = \text{arc purlin at } H.$$

$$648000'' : 13936'' : : 157.08' : 3'\ 4\frac{17}{32}'' = \text{arc purlin at } J.$$

$$13'\ 1\frac{1}{16}'' - 13'\ 0\frac{5}{8}'' = \frac{7}{16}'' = \text{excess of arc at } A.$$

$$12'\ 7\frac{23}{32}'' - 12'\ 7\frac{5}{16}'' = \frac{13}{32}'' = \text{excess of arc at } B.$$

$$11'\ 3\frac{15}{16}'' - 11'\ 3\frac{21}{32}'' = \frac{9}{32}'' = \text{excess of arc at } D.$$

$$9'\ 2\frac{29}{32}'' - 9'\ 2\frac{3}{4}'' = \frac{5}{32}'' = \text{excess of arc at } F.$$

$$6'\ 6\frac{3}{8}'' - 6'\ 6\frac{5}{16}'' = \frac{1}{16}'' = \text{excess of arc at } H.$$

$$3'\ 4\frac{17}{32}'' - 3'\ 4\frac{17}{32}'' = 0'' = \text{excess of arc at } J.$$

Example 23.—A Hemispherical Ring Dome without Trusses (Fig. 436)

342. Variations from Example 12 of Chapter IV.—Assume a dome of the same dimensions as in the last example, supported by 12 complete semicircular meridian ribs and 6 ring purlins, including the plate at base of dome, as in Fig. 436.

343. Lengths of Members.—The computations are made in the same manner as for the last example, omitting all those relating to the lower chord and web members.

344. Lengths of Purlin Chords and Arcs.—All lengths of panels of meridian rings will be the same as those previously found for panels of the upper chord, as well as those of the purlins. It only remains to obtain the lengths of diagonals for each area between two ribs and two purlins. Each area forms a trapezoid, whose meridian sides = chord length of a panel of meridian rib or of upper chord. Its horizontal sides are parallel and each = chord length of purlin on that side. Fig. 437 represents the development of these successive trapezoidal areas between two ribs.

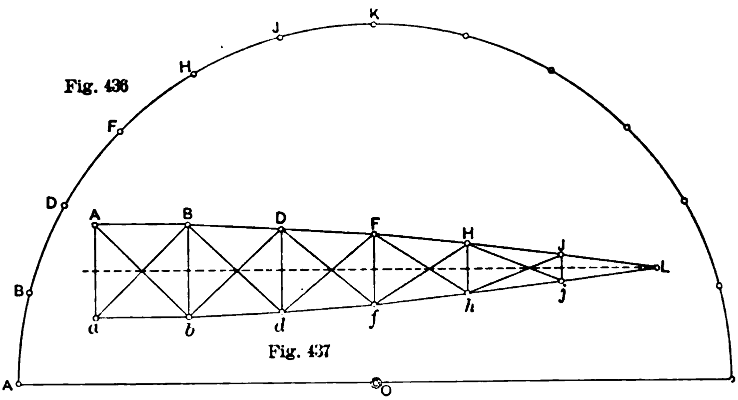


Fig. 436

Fig. 437

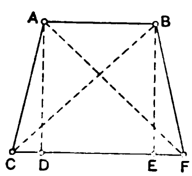


Fig. 438

345. Lengths of Diagonal Rods.—Let Fig. 438 represent one of these trapezoidal areas between chord panels $A\,C$, $B\,F$ of two ribs, and chords $A\,B$, $C\,F$ of two purlins.

$$C\,D = \frac{1}{2}\,(C\,F - A\,B). \quad D\,F = C\,F - C\,D.$$

$$A\,D = \sqrt{(A\,C^2 - C\,D^2)}. \quad A\,F = \sqrt{A\,D^2 + D\,F^2}.$$

Both diagonals of an area have the same lengths.

Take chord lengths of ribs and purlins previously computed in Example 21.

For figure $A\ B\ a\ b$, Fig. 437.

$$\frac{1}{2}\,(13'\,0\,\frac{5}{8}'' - 12'\,7\,\frac{5}{16}'') = 0'\,2\,\frac{21}{32}''.$$

$$13'\,0\,\frac{5}{8}'' - 0'\,2\,\frac{21}{32}'' = 12'\,9\,\frac{31}{32}''.$$

Perpendicular $= \sqrt{(13'\,0\,5/8'')^2 - (0'\,2\,13/16'')^2} = 13'\,0\,\frac{19}{32}''.$

Diagonal $= \sqrt{(13'\,0\,19/32'')^2 + (12'\,9\,13/16'')^2} = 18'\,3\,\frac{1}{2}''.$

For figure $B\,D\,b\,d$.

$$\frac{1}{2}\left(12'\,7\,\frac{5}{16}'' - 11'\,3\,\frac{21}{32}''\right) = 0'\,7\,\frac{27}{32}''.$$

$$12'\,7\,\frac{5}{16}'' - 0'\,7\,\frac{27}{32}'' = 11'\,11\,\frac{15}{32}''.$$

Perpendicular $= \sqrt{(13'\,0\,5/8'')^2 - (0'\,7\,27/32'')^2} = 13'\,0\,\frac{7}{16}''.$

Diagonal $= \sqrt{(13'\,0\,7/16'')^2 + (11'\,11\,15/32)^2} = 17'\,8\,\frac{1}{4}''.$

For figure $D\,F\,d\,f$.

$$\frac{1}{2}\left(11'\,3\,\frac{21}{32}'' - 9'\,2\,\frac{3}{4}''\right) = 1'\,0\,\frac{15}{32}''.$$

$$11'\,3\,\frac{21}{32}'' - 1'\,0\,\frac{15}{32}'' = 10'\,8\,\frac{3}{16}''.$$

Perpendicular $= \sqrt{(13'\,0\,5/8'')^2 - (1'\,0\,15/32'')^2} = 13'\,0\,\frac{1}{8}''.$

Diagonal $= \sqrt{(13'\,0\,1/8'')^2 + (10'\,8\,3/19'')^2} = 16'\,10''.$

For figure $F\,H\,f\,h$.

$$\frac{1}{2}\left(9'\,2\,\frac{3}{4}'' - 6'\,6\,\frac{5}{8}''\right) = 1'\,4\,\frac{1}{16}''.$$

$$9'\,2\,\frac{3}{4}'' - 1'\,4\,\frac{1}{16}'' = 7'\,10\,\frac{11}{16}''.$$

Perpendicular $= \sqrt{(13'\,0\,5/8'')^2 - (1'\,4\,1/16'')^2} = 12'\,11\,\frac{13}{16}''.$

Diagonal $= \sqrt{(12'\,11\,13/16'')^2 + (7'\,10\,11/16'')^2} = 15'\,2\,\frac{5}{16}''.$

For figure $H\,J\,h\,j$.

$$\frac{1}{2}\left(6'\,6\,\frac{5}{8}'' - 3'\,4\,\frac{17}{32}''\right) = 1'\,7\,\frac{1}{16}''.$$

$$6'\,6\,\frac{5}{8}'' - 1'\,7\,\frac{1}{16}'' = 4'\,11\,\frac{9}{16}''.$$

$$\text{Perpendicular} = \sqrt{(13'\,0\,5/8'')^2 - (1'\,7\,1/16'')^2} = 12'\,11\frac{15}{32}''.$$

$$\text{Diagonal} = \sqrt{(12'\,11\,15/32'')^2 + (4'\,11\,9/16'')^2} = 13'\,10\frac{15}{32}''.$$

346. Construction of Ring Dome.—In constructing a ring dome, it will be more convenient and economical to rivet lower flange of purlin on upper flange of meridian rib, instead of cutting purlins between ribs, so that the centre line of the purlin would meet centre lines of ribs at the apexes. Then the chord and arc lengths of the purlin are both increased, since they are more distant from the centre of the dome by half (depth of rib + depth of purlin).

For example, let $10''$ = depth of rib and $8''$ = depth of purlin. Then $\frac{1}{2}(10'' + 8'') = 9'' = 0.75$ ft. = increase in the radii drawn to ends of axis of purlin, making total radius = 50.75 ft.

347. Increased Lengths of Purlins on Upper Chord.—Since these purlin axes are parallel to each other between two adjacent apexes, and the angles at the centre subtended by them are identical, the two triangles formed by the end radii and the parallel purlin axes are similar. Therefore the chord length of the inner axis will be increased by $\frac{0.75'}{50'} = 0.015$ of its original length.

Hence, purlin at $A = 13'\,0\frac{5}{8}'' \times 1.015 = 13'\,2\frac{31}{32}''$.

$$\text{Purlin at B} = 12'\,7\frac{5}{16}'' \times 1.015 = 12'\,9\frac{9}{16}''.$$

$$\text{Purlin at } D = 11'\,3\frac{21}{32}'' \times 1.015 = 11'\,5\frac{11}{16}''.$$

$$\text{Purlin at } F = 9'\,2\frac{3}{4}'' \times 1.015 = 9'\,4\frac{13}{32}''.$$

$$\text{Purlin at } H = 6'\,6\frac{5}{8}'' \times 1.015 = 6'\,7\frac{5}{16}''.$$

$$\text{Purlin at } J = 3'\,4\frac{17}{32}'' \times 1.015 = 3'\,5\frac{5}{32}''.$$

As these purlins are curved in horizontal circles and not in a great circle of the sphere, it becomes necessary to determine the horizontal radii to their ends before computing their arc lengths.

348. Radii of Purlins.—

Thus $50'\,0'' \times 1.015 = 50'\,9''$ = horizontal radius at A.

$$48'\,3\frac{9}{16}'' \times 1.015 = 49'\,0\frac{1}{4}'' \text{ at } B.$$

$$43'\,3\frac{5}{8}'' \times 1.015 = 43'\,11\frac{13}{32}'' \text{ at } D.$$

$$35'\,4\frac{9}{32}'' \times 1.015 = 35'\,10\frac{5}{8}'' \text{ at } F.$$

$$25'\,0'' \times 1.015 = 25'\,4\frac{1}{2}'' \text{ at } H.$$

$$12'\,11\frac{9}{32}'' \times 1.015 = 13'\,1\frac{1}{2}'' \text{ at } J.$$

Then since the sectors are similar:

349. Lengths of Purlin Arcs.—

$$50'\,0'' : 50'\,9'' :: 13'\,0\frac{5}{8}'' : 13'\,2\frac{31}{32}'' = \text{arc purlin at } A.$$

$$48'\,3\frac{9}{16}'' : 49'\,0\frac{1}{4}'' : 12'\,7\frac{5}{16}'' : 12'\,9\frac{9}{16}'' = \text{arc purlin at } B.$$

$$43'\,3\frac{5}{8}'' : 43'\,11\frac{13}{32}'' :: 11'\,3\frac{21}{32}'' : 11'\,5\frac{11}{16}'' = \text{arc purlin at } D.$$

$$35'\,4\frac{9}{32}'' : 35'\,10\frac{5}{8}'' :: 9'\,2\frac{3}{4}'' : 9'\,4\frac{13}{32}'' = \text{arc purlin at } F.$$

$$25'\,0'' : 25'\,4\frac{1}{2}'' :: 6'\,6\frac{5}{16}'' : 6'\,7\frac{1}{2}'' = \text{arc purlin at } H.$$

$$12'\,11\frac{9}{32}'' : 13'\,1\frac{1}{2}'' :: 3'\,4\frac{9}{16}'' : 3'\,5\frac{1}{8}'' = \text{arc purlin at } J.$$

CHAPTER VIII

STABILITY OF THE SUPPORTS OF ROOFS UNDER WIND PRESSURE

350. Need for Consideration.—In the previous consideration of the roof trusses, no attention has been paid to the manner in which they are supported at their ends, excepting the use of pin joints or expansion rollers. It is very evident that a truss might support its loads with entire safety, yet the structure might be destroyed by the failure of the supporting walls or columns under the maximum wind pressure acting on them and the roof. It therefore becomes necessary to study the general conditions of stability against wind pressures on structures, and to carefully examine some typical cases.

351. Masonry Wall under Horizontal Wind Pressure. (Fig. 439.)—For simplicity, commence with a masonry wall of uniform thickness exposed to a horizontal wind pressure.

Let Fig. 439 represent such a wall with thickness t and height h in ft., built of masonry weighing w lbs. per cubic ft., and able to resist a safe compression of y lbs. per sq. ft.

Let p = wind pressure in lbs. per sq. ft. against side of wall.

Consider one foot in length of the wall.

Weight of wall $= h\,t\,w$ in lbs., or $\dfrac{h\,t\,w}{2000}$ in tons.

Wind pressure $= p\,h$ in lbs., or $\dfrac{p\,h}{2000}$ in tons.

The base or thickness of the wall is divided into three equal parts as indicated by the semicircles in Figs. 439 to 443.

Six different cases may occur.

352. Case 1. No Wind Pressure.—Vertical resultant of weight of wall intersects its base at middle, coinciding with vertical axis of the wall. Uniform pressure of wall on its base $= h\,w$ per sq. ft.

353. Case 2. Wind Pressure Light.—Resultant of weight of wall and of wind pressure falls within the middle third of its base. (Fig. 440.)

The weight of the wall in tons is laid off from its centre C to a, then the total wind pressure in tons from a to b. Join $C\,b$ and

produce to intersect base at d. The resultant R may be resolved at d into a horizontal component $a\ b$ and a vertical component $C\ a$. The horizontal component tends to slide the wall in the direction of the wind, but this may generally be neglected, on account of the great friction of masonry on masonry. The vertical component

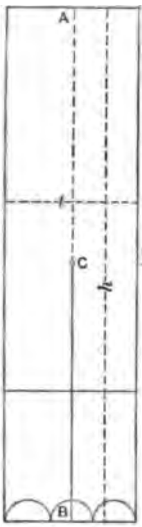


Fig. 439

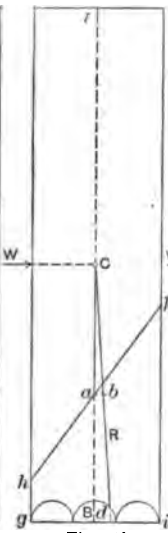


Fig. 440

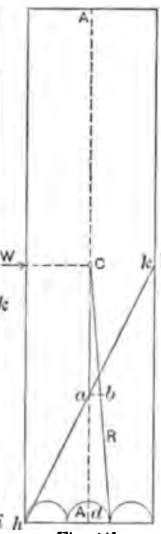


Fig. 441

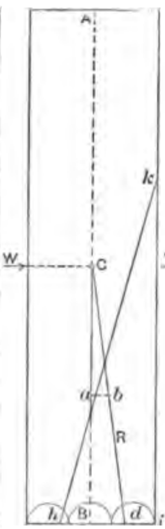


Fig. 442

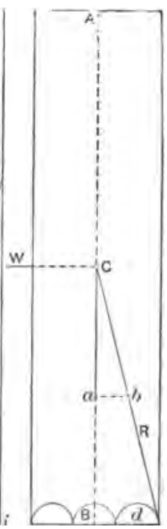


Fig. 443

acts vertically at d, nearest the leeward side of the wall, so that this vertical pressure is not uniformly distributed, being greatest at the leeward edge of base and smallest at its windward edge. These intensities may be computed by the following formulas.

Let f = eccentricity $B\ d$ in ft., or distance from middle of wall to d.

Then $h\ w\ (1 + \frac{6f}{t})$ = maximum intensity at leeward edge.... (37)

And $h\ w\ (1 - \frac{6f}{t})$ = minimum intensity at windward edge.. (38)

Lay off these intensities $g\ m$ and $i\ k$ in Fig. 440; join $m\ k$, and the pressure area $m\ g\ i\ k$ represents the varying intensity of pressure of the wall on its base, as well as its total pressure. This area equals the rectangular pressure area in Fig. 439, and its centre of gravity must be vertically over d.

354. Case 3. Wind Pressure Safe.—Resultant falls at edge of middle third of base. (Fig. 441.)

Since the centre of gravity of the pressure area must be vertically over d, this area must be a triangle, thus making 0 pressure at windward edge and one of $2\,h\,w$ at leeward edge.

355. Case 4. Wind Pressure Heavy.—Resultant falls outside middle third of base. (Fig. 442.)

The pressure area must still be a triangle with its centre of gravity vertically over d. Hence make $m\,i$ thrice $d\,i$ and construct an area = rectangle in Fig. 439. Its altitude $= i\,k = \dfrac{2\,h\,w\,t}{m\,i} \ldots (39)$

Since it is unsafe to consider the tensile resistance of masonry, on account of the possible lack of adhesion of mortar at a joint, the base joint of the wall is assumed to open slightly as far as m, where 0 compression occurs from O to the windward edge, but not tension.

This condition of the wall is still safe, provided the leeward intensity of pressure $i\,k$ does not exceed the safe limit of resistance to compression for the kind of masonry employed. But if this be exceeded, the masonry will be in danger of crushing, and if the wind pressure increases, the intensity of compression increases far more rapidly until the wall fails by crushing at its leeward edge before being directly overturned about this edge.

356. Case 5. Wind Pressure Destructive.—The resultant falls at leeward edge of base. (Fig. 443.)

The wall is in unstable equilibrium, being on the point of overturning about its leeward edge d. But since the intensity of compression at d is then infinitely great, it would certainly fail by crushing before overturning.

357. Case 6. Wind Pressure a Hurricane.—Resultant falls outside the base.

The wall must have already failed by crushing and overturning.

358. Results of this Examination.—A careful study of these different cases shows that the resultant R may fall outside the middle third of the base without danger of failure for the wall, so long as the maximum intensity of pressure does not exceed the safe resistance of the masonry. To keep the resultant within the middle third of the base merely signifies that the maximum intensity shall not exceed twice the uniform intensity of compression. There is nothing sacred in the principle of the middle third. This explanation equally applies to masonry arches, vaults, domes, chimneys, etc.

Example 1.—A Masonry Wall. (Figs. 444 to 446)

359. Description.—Let this wall have the following dimensions, weights, and resistances.

Wall 19 ft. high and 3 ft. thick, built of common bricks in lime mortar, weighing 112 lbs. per cu. ft.; maximum safe resistance to compression = 10000 lbs., or 5 tons per sq. ft.

Weight of one lineal ft. of wall = 19 × 3 × 112 = 6384 lbs.

Then $\frac{6384}{3} \doteq 2128$ lbs. = uniform intensity of pressure on foundation, less than one-fourth its safe resistance.

360. Wind Pressure Light.—Assume wind pressure of 30 lbs. per sq. ft. on its vertical surface.

Total pressure = 30 × 19 = 570 lbs. (Fig. 444.)

Lay off $C\,a$ = 6384 lbs. and $a\,b$ = 570 lbs.; join $C\,b$ and produce resultant R to d; lay off thrice $d\,i$ from i to m to obtain base of pressure area. Maximum intensity $i\,k$ = $\frac{2 \times 6384}{1.95}$ = 6535 lbs. Lay this off from i to k and join $m\,k$, when $m\,k\,i$ = required pressure area. The joint opens as far as m; the resultant R falls without the middle third, but since the maximum intensity does not exceed 62 per cent of the safe allowable compression on the masonry, the wall is perfectly safe under this wind pressure.

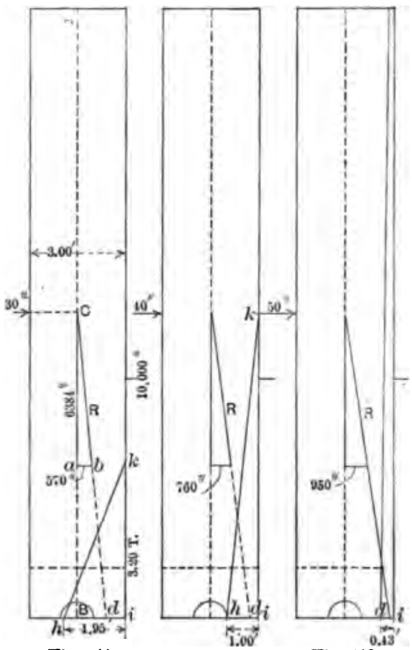


Fig. 444 Fig. 445 Fig. 446

361. Wind Pressure Medium.—Assume a wind pressure of 40 lbs. per sq. ft. (Fig. 445.)

Total wind pressure = 19 × 40 = 760 lbs. Proceed as before to locate $d\,d$ and make $i\,m$ = thrice $i\,d$, which is 1.00 ft. Hence $\frac{2 \times 6384}{1.00}$ = 12768 lbs. to be laid off to k; join $m\,k$. This intensity

at leeward edge exceeds safe intensity of compression by nearly 28 per cent, so that the wall is unsafe.

362. Wind Pressure Maximum.—Assume a wind pressure of 50 lbs. per sq. ft. (Fig. 446.)

Total wind pressure = 19 × 50 = 950 lbs. By the previous method, $m\,i$ is found = 0.43 ft. And $i\,k = \frac{2 \times 6384}{0.43} = 29693$ lbs. = leeward intensity. This leeward intensity greatly exceeds the safe resistance of the masonry to compression, so that the wall would already have been destroyed under a smaller wind pressure.

Therefore, if this wall were exposed to a wind pressure exceeding 35 lbs. per sq. ft., its thickness must be increased to make it safe.

Example 2.—Two Walls are Connected Horizontally at Top by Struts, etc. (Fig. 447)

363. Description.—Each wall is 19 ft. high and 3 ft. thick, built of hard bricks in cement mortar. Weight 125 lbs. per cu. ft. Safe resistance to compression 25000 lbs. per sq. ft. Wind pressure 50 lbs. per square foot.

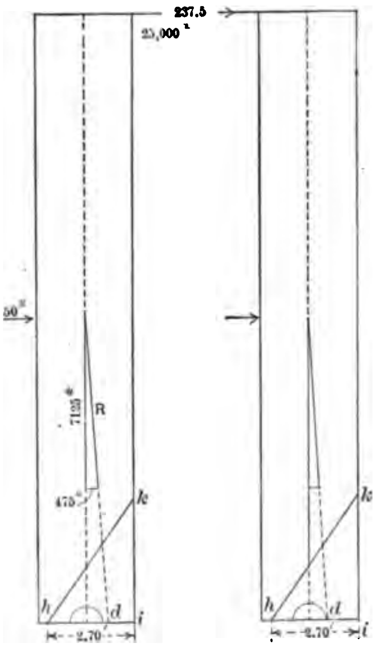


Fig. 447

Consider one foot in length of each wall.

Weight of one wall = 19 × 3 × 125 = 7125 lbs.

Wind pressure on windward wall = 19 × 50 = 950 lbs.

364. Resistance to Wind Pressure. — Half this pressure should be resisted by each wall, its share being transmitted to leeward wall by struts connecting the tops of the walls. Hence $\frac{950}{2} = 475$ lbs. = pressure on one wall. Proceeding as before, $i\,m$ is found = 2.70 ft. Then $\frac{2 \times 7125}{2.70} = 5278$ lbs. per sq. ft. = intensity of compression at leeward edge of wall. Although the resultant R intersects the base outside its middle third, the

walls are amply safe, since the maximum intensity of compression barely exceeds 21 per cent of its safe intensity.

If both walls equally resist the wind pressure and are properly connected at top, the force transmitted thereby $= \frac{950}{4} = 237.5$ lbs. per lineal foot of the wall. If these connections were roof trusses 16 ft. on centres, the force transmitted by each truss $= 237.5 \times 16 =$ 3800 lbs., which would produce compression to that amount in a horizontal lower chord or more in one cambered or raised at the centre. This would seriously lessen the usual tension in lower chord and might even exceed it, causing a reversed stress, requiring its effects to be investigated.

Example 3.—Wall Supporting a Gable Roof (Fig. 448)

365. Description.—A single masonry wall supports a gable roof resting on cantilever trusses, forming a structure suitable for suburban railway station, etc. Roof 19 ft. span and 4 ft. rise, resting on wall of hard bricks in cement, 3 ft. thick and 8 ft. high from top of platform, or 12 ft. above foundation. Platform is directly supported by the ground.

Consider one lineal foot of wall and roof.

Permanent load is found to be 483 lbs.

Snow load is computed at 228 lbs.

Wind load is here 208 lbs.

Weight of wall $= 12 \times 3 \times 125 = 4500$ lbs.

Wind pressure on wall $= 8 \times 40 = 320$ lbs.

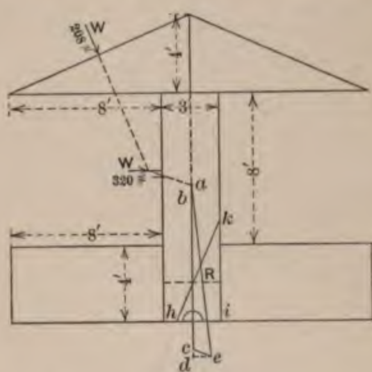


Fig. 448

366. Resistance to Wind Pressure.—A cross-section of the wall, roof, and platforms is given in Fig. 448. The resultant of wind pressures on roof and wall is found, then produced to intersect vertical axis of wall at a, from which are laid off sum of P and S loads to b, weight of wall to c, and resultant of wind loads to e. The resultant of all loads and forces acting on wall is $a\ e$; drawing horizontal $e\ d$, $a\ d$ = 5200 lbs. = total vertical component or load on base

of wall. And $\frac{5200}{3}$ = 1733 lbs. = uniform intensity of pressure on base.

The resultant R cuts the base outside its middle third at 0.80 ft. from edge, making base of pressure area = 3 × 0.8 = 2.40 ft. Hence its altitude = $\frac{2 \times 5200}{2.40}$ = 4333 lbs. = intensity of compression at leeward edge of base. This structure is entirely safe, the intensity being 18 per cent of the safe intensity.

Example 4.—Masonry Piers Supporting a Gable Roof (Fig. 449)

367. Description.—The same roof is supported by piers of similar masonry 12 ft. high and 4 ft. square, set 12 ft. on centres.

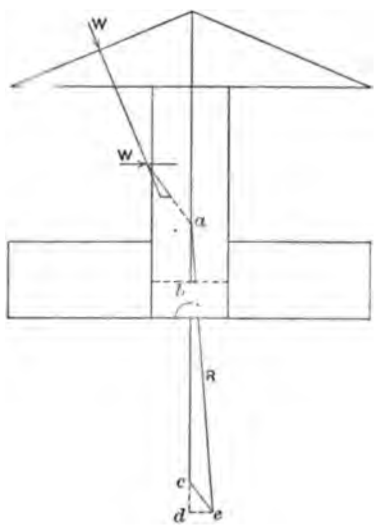


Fig. 449

Weight of one pier = 12 × 4 × 4 × 125 = 24000 lbs. = 12.00 tons. P and S loads on roof = 711 × 12 = 8532 lbs. = 4.27 tons.

W load on roof = 208 × 12 = 2496 lbs. = 1.248 tons.

Wind pressure on pier = 8 × 4 × 40 = 1280 lbs. = 0.64 ton.

368. Resistance to Wind Pressure. — Obtaining resultant of wind pressures on roof and pier, this is produced to cut axis of pier at a, P and S loads are laid off to b, then $b\ c$ = weight of pier, and horizontal $e\ d$ makes vertical component of R = 18.20 tons. Hence $\frac{18.20}{16}$ = 1.1375 tons per sq. ft. = uniform intensity of compression. Since the resultant falls within the middle third of base, the maximum intensity must be computed by formula. Eccentricity = 0.4 ft.

$$1.1375\left(1 + \frac{6 \times 0.4}{4}\right) = 1.82$$ tons per sq. ft., which is amply safe, the safe resistance being 12.5 tons per sq. ft. Indeed, the side

of the pier might be reduced to 3′ 8″ and perhaps less with safety. This type of structure is then preferable to the last and is more economical.

EXAMPLE 5.—STEEL COLUMNS SUPPORTING A GABLE ROOF (Fig. 450)

369. Description.—The same roof and its loads are supported by single steel columns set 16 ft. on centres and each is firmly anchored by extending down into a concrete block foundation. Each is composed of two 6″, 10.5 = lb. channels and two 8″ × 5/16″ plates, weight about 600 lbs. = 0.30 ton.

$$P \text{ and } S \text{ load of roof} = \frac{4.266 \times 16}{12} = 5.69 \text{ tons.}$$

$$W \text{ load on roof only} = \frac{2.258 \times 16}{12} = 3.00 \text{ tons.}$$

The resultant R is found graphically = 8.80 tons. It may be resolved into a V component of 8.71 tons and an H component of 1.20 tons.

370. Resistance to Wind Pressure.—Assuming a maximum safe pressure of the concrete block on the soil at 1.50 tons per sq. ft., and assuming a concrete block footing 4 ft. square and 2 ft. deep, its weight $= \frac{4 \times 4 \times 150 \times 2}{2000} = 2.40$ tons.

Uniform pressure on soil $= \frac{8.71 + 2.40}{16} = 0.695$ tons per sq. ft.

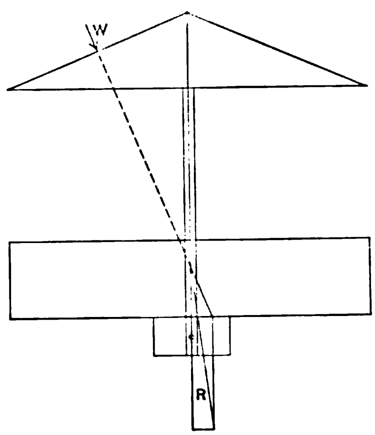


FIG. 450

Eccentricity of resultant at bottom of block = 0.65 ft.

Then $0.695\left(1 + \frac{6 \times 0.65}{4}\right) = 1.375$ tons per sq. ft. = maximum intensity.

$0.695\left(1 - \frac{6 \times 0.65}{4}\right) = 0.014$ tons per sq. ft. = minimum intensity.

This base would then be safe on the soil.

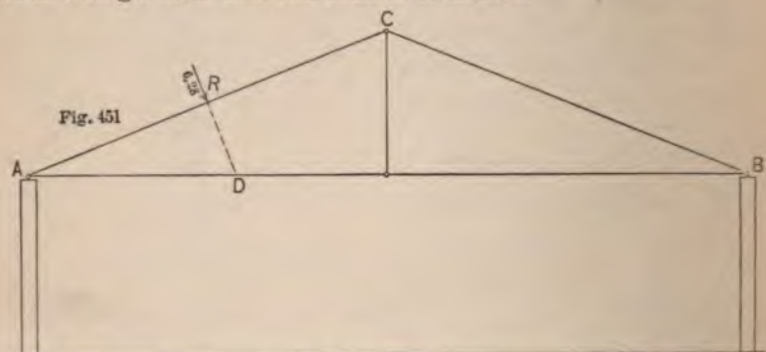
Lever arm of R about centre of concrete block = 0.50 ft. Its moment $M = 0.50 \times 8.80 = 4.40$ ft. tons.

Since $s = \frac{Mc}{I} = \frac{4.40 \times 1}{2/3 \times 2^3} = 0.826$ tons per sq. ft., = maximum pressure of steel column against concrete at top and bottom of block; this is entirely safe after the concrete has properly set. Therefore concrete footing $4 \times 4 \times 2$ ft. is amply sufficient in this case. It is further at once evident that the best mode of supporting the roof is by steel columns. These can be strongly fixed in the concrete base by means of horizontal steel beams riveted to the columns and embedded in the concrete.

EXAMPLE 6.—MASONRY WALLS SUPPORTING A GABLE ROOF (Fig. 451)

371. Description.—Ends of the truss are anchored to each wall at top. Roof 100 ft. span; 20 ft. rise; walls 25 ft. high and 2 ft. thick, of good bricks in lime mortar; trusses 16 ft. on centres and of steel; tin on longleaf pine sheathing, rafters and purlins; no ceiling; wind pressure 30 lbs. per sq. ft.; masonry weighs 112 lbs. per cu. ft. Effect of wind pressure appears to be greatest if the snow load be omitted. It is here assumed that a strong wooden plate is anchored to the top of the wall by long bolts, and that this together with the horizontal strength of the wall may be assumed to cause the wall to act as a single mass between centres of trusses.

Fig. 451



372. Dimensions.—Angle of inclination of roof = 21.8°.
Length of slope = $\sqrt{50^2 + 20^2} = \sqrt{2900} = 53.85$ ft.
Wall surface between centres of trusses = $25 \times 16 = 400$ sq. ft.
Surface of one side of roof = $53.85 \times 16 = 861.6$ sq. ft.

For convenience it may here be assumed that one truss supports an entire bay of the roof, which is practically correct.

373. Loads on Roof.—

$$\text{Weight of truss} = \frac{4.794 \text{ lbs.} \times 16 \times 10}{2000} = 3.835 \text{ tons.}$$

$$\text{Total } P \text{ load} = \frac{(2 + 4 + 4 + 3) \times 16 \times 2 \times 53.85}{2000} = 11.201 \text{ tons.}$$

$$W \text{ load on roof} = \frac{14.53 \times 16 \times 53.85}{2000} = 6.28 \text{ tons.}$$

Locating resultant R of wind pressure on roof in Fig. 451 and extending it to cut the span line in D, we find $A\,D = 29$ ft., and $D\,B = 71$ ft.

Then $6.28 \times \frac{71}{100} = 4.445$ tons supported at A.

$6.28 \times \frac{29}{100} = 1.815$ tons supported at B.

$\frac{3.835 + 11.201}{2} = 7.8518$ tons $= P$ loads at A and B.

$$\text{Wind pressure on wall} = \frac{30 \times 16 \times 25}{2000} = 5.800 \text{ tons.}$$

$$\text{Weight of one wall} = \frac{25 \times 16 \times 2 \times 112 \text{ lbs.}}{2000} = 44.800 \text{ tons.}$$

374. Division of Wind Pressure Between Walls.—Normal wind pressure on the roof is divided between supports A and B inversely as span line is divided by resultant R.

Horizontal wind pressure on windward wall is divided equally between both walls, this being the principal force tending to overthrow them, and both resist it equally or must fall together.

Then $\frac{6.28 \sin 21.8°}{2} = 1.163$ tons $=$ horizontal component of normal wind pressure acting at top of each wall.

Assume that this horizontal component be transferred to act at mid-height of the wall and combined with the resultant of wind pressure on wall, where it would evidently be doubled to produce an equal moment at the base.

Then $\frac{59900}{2} + 2 \times 1.635 = 5.275$ tons $=$ total H wind pressure acting at mid-height of each wall. (Fig. 452.)

And 1.815 sin 21.8° = 0.674 ton = horizontal force directly applied at B. $\frac{5.275}{2}$ = 2.638 tons = horizontal force acting at top of wall B.

Then 2.638 − 0.674 = 1.964 tons to be transmitted through the truss, producing uniform compression in the lower chord if this be horizontal, thereby reducing the tensile stress in each of its members. Should this transmitted stress exceed the tension in any member of the lower chord, which is not probable, this excess becomes an additional horizontal force, doubled at mid-height, to be resisted by the windward wall.

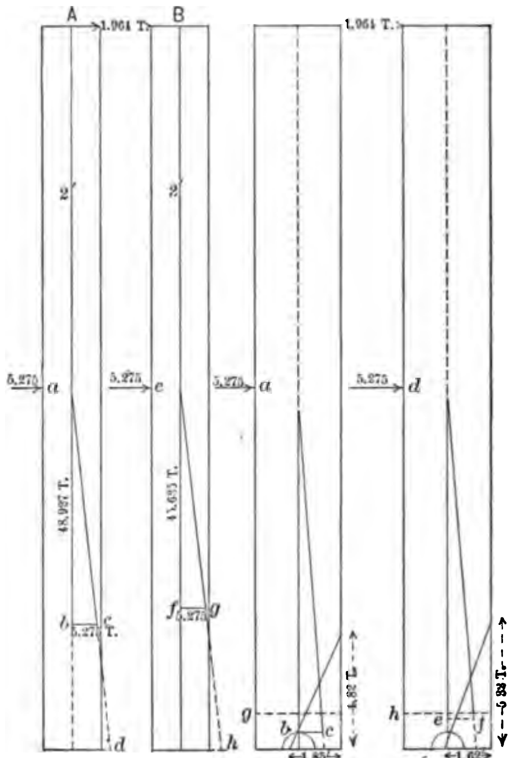


Fig. 452 Fig. 453

375. Loads on Walls.—Also 4.445 cos 21.8° = 4.127 tons = V component at A.
1.815 cos 21.8° = 1.685 tons = V component at B.

Total V load $= 44.800 + 4.127 = 48.927$ tons for windward wall.

Total V load $= 44.800 + 1.685 = 45.685$ tons for leeward wall.

376. Walls Two Feet Thick; Unsafe.—Obtaining in Fig. 452 the resultant $a\,c$ for the windward wall and $e\,f$ for the leeward wall, both fall entirely outside the base of each wall, showing that the thickness of the walls must be materially increased, or buttresses must be added beneath the ends of the truss.

377. Walls Three Feet Thick; Safe.—Assume the thickness to be made 3 ft., as in Fig. 453.

Wind loads are the same as before.

Weight of one wall $= 44.800 \times \frac{36}{24} = 67.200$ tons.

Hence $67.200 + 4.127 = 71.327$ tons $= V$ load for windward wall.
$67.200 + 1.685 = 68.885$ tons $= V$ load for leeward wall.

In Fig. 453, making $a\,b = 71.327$ tons and $b\,c = 5.275$ tons, their resultant $a\,c$ cuts the base considerably outside its middle third. Also making $d\,e = 68.885$ tons and $e\,f = 5.275$ tons, their resultant $d\,f$ falls still nearer the edge of the base of leeward wall.

Then $\frac{71.327}{16 \times 3} = 1.485$ tons = uniform intensity of V load per sq. ft. of base of windward wall, when no wind exists.

$\frac{68.885}{16 \times 3} = 1.435$ tons = uniform intensity of V load per sq. ft. of base of leeward wall.

These intensities are laid off at a much larger scale than that used for obtaining the resultants, and the horizontal dotted line encloses the rectangular pressure area, numerically = total V load on base.

Next laying off from leeward edge of each wall thrice the distance to its resultant, the base of the triangular pressure area is found.

$\frac{1.485 \times 3 \times 2}{1.85} = 4.82$ tons = altitude of pressure triangle and = maximum intensity of compression on base of windward wall.

$\frac{1.435 \times 3 \times 2}{1.62} = 5.32$ tons = altitude of pressure triangle and = maximum intensity of compression on base of leeward wall.

The pressure area triangles are then drawn in the figure. Since the maximum intensity of compression is 5.32 tons per square foot

and the Chicago ordinance allows 9 tons for this kind of masonry, these walls are amply safe, unless exposed to a wind pressure exceeding 30 lbs. per sq. ft.

378. Walls with Buttresses more Economical.—It is here sufficiently evident that masonry walls supporting large roofs are frequently made dangerously thin to safely resist wind pressures.

But it would be more economical to make the walls considerably thinner with a heavy plaster or buttress under the end of the truss on each. (Fig. 454.)

Fig. 454

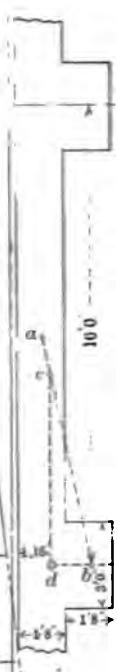


Fig. 455

Assume 1′ 8″ for thickness of the walls with pilasters 3 ft. wide and projecting internally 1′ 8″, making a total of 3′ 4″.

First locate the centre of gravity of the wall and buttress. Fig. 455 represents the plan of a part of the wall and two buttresses. The centre of gravity of wall is at a, that of the pilaster at b. $a\,b$ is divided inversely at c as the relative areas of wall and buttress, so that the centre of gravity of one bay of the whole is at d, 1.16′ from outside of wall.

Weight of wall and one pilaster

$$= \frac{31\,2\;\;3 \times 25 \times 112}{2000} = 44.333$$

tons.

$44.333 + 4.127 = 48.460$ tons = vertical component for windward wall.

$44.333 + 1.685 = 46.018$ tons = vertical component for leeward wall.

The horizontal wind forces acting at mid-height = 5.275 for each wall.

Making $a\,b$ = 48.46 tons and $d\,e$ = 46.018 tons, also $b\,c$ and $e\,f$ = 5.275 tons, their resultants $a\,c$ and $d\,f$ are drawn, showing that the windward wall is probably stable, while the leeward wall would certainly fall, as its resultant falls outside its base.

379. Buttresses Should Project Externally and Internally.— Therefore it appears that the pilasters should project both externally and internally. Assume walls 1′ 8″ thick with pilasters 3 ft. wide and projecting one foot on exterior and interior. (Fig. 456.)

Weight of wall is found to be 91,467 lbs. = 45.734 tons.

Then

$45.734 + 4.127 = 49.861$ tons $= V$ component for windward wall.

$45.734 + 1.685 = 47.619$ tons $= V$ component for leeward wall.

$\frac{49.861}{32\,2/3} = 1.526$ = uniform intensity of pressure on base of W wall.

$\frac{47.619}{32\,2/3} = 1.458$ = uniform intensity of pressure on base of L wall.

These uniform intensities of pressure are indicated in Fig. 456 by the dotted horizontal lines at a larger scale. The gravity axis here coincides with the centre of each wall. Resultants of V and H components are obtained as before and are produced to cut the base of each wall. The base of the pressure triangles are found as before, the points of O pressure being marked O. For the windward wall, since the point O falls at the middle of the wall, intensity at inner edge of wall = half that at inner edge of pilaster.

Let x = maximum intensity of pressure in tons per sq. ft. at inner edge of pilaster.

Volume of pressure wedge on base of wall only $= 16 \times 1 \times \frac{x}{4} = 4\,x$.

Volume of prism frustum of pressure on base of pilaster only $= 1 \times 3 \times \frac{3\,x}{4} = 2.25\,x$.

Hence the total volume of pressure solid $= 4\,x + 2.25\,x = 6.25\,x$.

This volume must equal the total V component acting on base of wall.

Then $6.25\,x = 49.861$ tons. And $x = \frac{49.861}{6.25} = 7.90$ tons per sq. ft. = intensity of pressure at inner edge of inner pilaster.

For the leeward wall, the point O falls outside the middle vertical and maximum intensity of pressure at outer edge of wall $= 0.49\,x$. Width of pressure base for wall = 0.92 ft.

Volume of pressure wedge for wall only $= 16 \times 0.92 \times \frac{0.49\,x}{2} = 3.60\,x$.

Volume of pressure prism for pilaster $= 3 \times 1 \times \frac{(0.49 + 1.00)x}{2} =$ $2.24\ x$.

Total pressure volume $= 5.84\ x = 47.619$ tons, and $x = 8.16$ tons per sq. ft.

These walls are then safe, since neither maximum intensity exceeds 9 tons, the safe limit for the kind of masonry employed.

380. Conclusions Relating to Walls Supporting Roofs.—This example clearly indicates the following conclusions:

1. Walls are more economical if built with pilasters under the ends of roof trusses, than if made of uniform thickness and without them.

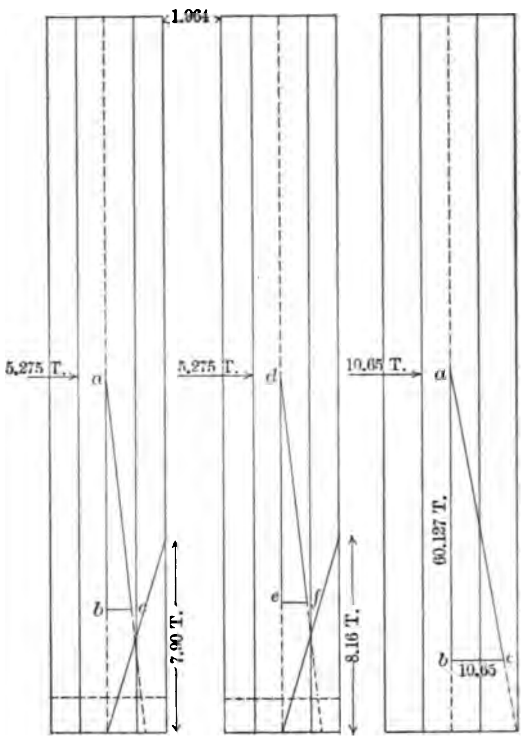


Fig. 456 Fig. 457

2. Such pilasters should project both internally and externally.

3. Buildings with trussed roofs of wide span frequently have dangerously thin walls.

Example 7.—Same Roof with Expansion Rolls under one End of Truss (Fig. 457)

381. Description.—If the rolls be at the windward wall, that wall must alone resist the entire horizontal wind pressure acting on it. All the horizontal components of the wind pressure on the roof must be resisted by the leeward wall alone. If the rolls be at the leeward wall, both the wind pressure on the windward wall and the entire horizontal component of wind pressure on the roof must be resisted by the windward wall alone. Therefore the latter is the more dangerous condition of the structure and will alone be considered. But both walls must be of equal thickness and resistance, unless the leeward be sheltered by an adjacent building, etc.

382. Rolls at Windward Wall.—Assuming expansion rolls at windward wall. Try walls 2 ft. thick with pilasters 3 ft. wide and projecting 1′ 4″ externally and internally.

Weight of one wall = $40 \times 25 \times 112 = 112000$ lbs. = 56.00 tons per bay.

Wind pressure on wall = $16 \times 25 \times 30 = 12000$ lbs. = 6.00 tons.

Normal wind pressure on roof = 6.26 tons as before.

Then $6.26 \sin 21.8° = 2.325$ tons = horizontal component of wind pressure.

This is applied at top of wall and $= 2.325 \times 2 = 4.65$ tons at mid-height.

Then $6.00 + 4.65 = 10.65$ tons = H wind component for windward wall.

And $4.127 + 56.00 = 60.127$ tons = V component for the same.

These components are laid off in Fig 457, obtaining their resultant $a\ c$, which cuts the base but slightly within the internal pilaster. The assumed thickness would then be unsafe, since the intensity of compression at inside edge of pilaster would greatly exceed 9.00 tons per sq. ft.

Assume a wall 2′ 4″ thick with buttresses projecting 1′ 8″. (Fig. 458.)

$$\text{Area of base} = 16' \times (2'\,3'') + 3' \times (1'\,8'') = 47\frac{1}{3} \text{ sq. ft.}$$

$$\text{Weight of wall} = 47\frac{1}{3} \times 25 \times 112 = 132533 \text{ lbs.} = 66.267 \text{ tons}$$

Then $66.267 + 4.127 = 70.394$ tons = total V component.

And $\frac{70.394}{47\ 1/3} = 1.487$ tons per sq. ft. = uniform intensity of pressure on base.

The 0 compression here occurs at 0.10 ft. from centre of the wall.

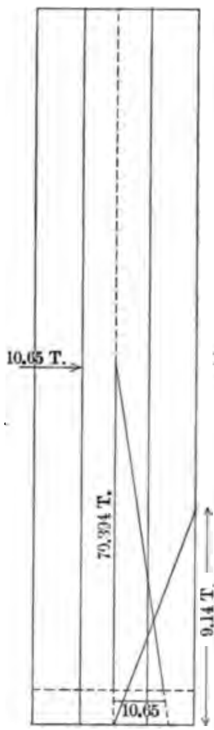


Fig. 458

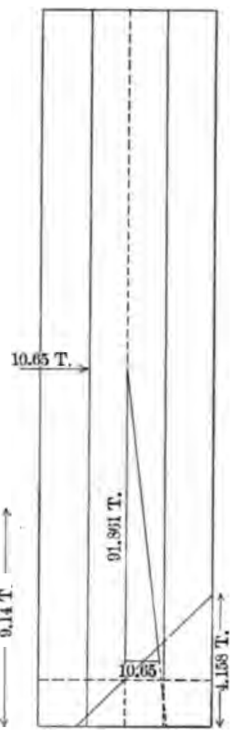


Fig. 459

Let x = maximum intensity of compression at edge of buttress.

Volume of pressure solid for wall $= 16 \times 1.1 \times \frac{0.46\,x}{2} = 4.048\,x.$

Volume of pressure solid for buttress $= 3 \times 1.67 \times \frac{0.46 + 1.00\,x}{2} = 3.650\,x.$

Total volume of pressure solid $= x\,(4.048 + 3.650) = 7.698\,x.$

Equating: $7.698\,x = 70.394$ tons.

$x = 9.14$ tons per sq. ft.

This intensity of pressure slightly exceeds the safe resistance of the masonry.

Increase thickness of wall from 2′ 4″ to 2′ 8″; buttresses as before.

Area of base $= 16 \times 2\frac{2}{3} + 4 \times 3 \times 1\frac{2}{3} = 62\frac{2}{3}$ sq. ft.

Weight of wall $= 62\frac{2}{3} \times 25 \times 112 = 17546$ lbs. $= 87.734$ tons.

Then $87.734 + 4.127 = 91.861$ tons = total V component on base.

Uniform intensity of pressure on base $= \dfrac{91.861}{62\,2/3} = 1.466$ tons per sq. ft.

Volume pressure solid for wall $= 16 \times 3.00 \times \dfrac{0.74\,x}{2} = 17.74\,x$.

Volume pressure solid for buttress $=$ $3 \times 1.67 \times \dfrac{0.74+1.00}{2} \times x = 4.35\,x$.

Then $17.74\,x + 4.35\,x = 22.09\,x =$ 91.861 tons.

Hence $x = 4.158$ tons per sq. ft. Which is less than one-half the safe resistance of the masonry and therefore amply safe.

383. Expansion Rolls Make Thicker Walls Necessary.—The last example plainly shows that masonry walls must be made much thicker, when expansion rolls are placed under one end of the truss, as the walls are then unable to help each other resist the wind pressure.

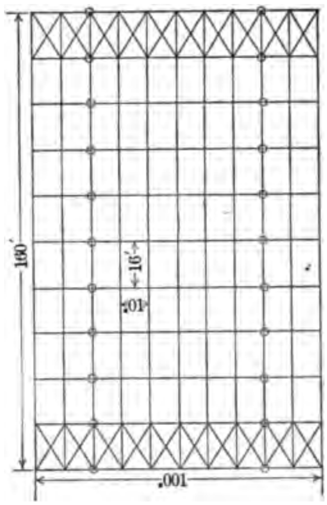


Fig. 460.

Example 8.—An Open Train Shed

384. Description.—Fig. 460 is a complete plan of the roof, indicating the trusses, purlins, horizontal ceiling trusses in end bays, and the locations of two rows of supporting columns.

Fig. 461 is a cross-section of the structure without side or end walls, excepting for the closed gable ends. Two columns support each truss, being connected therewith by external diagonals or braces. The columns are 18 ft. high and the diagonals are attached at 8 ft. above the ground, so that the lower portion of the column is subjected to a large bending moment, when the wind acts on the

roof. Construction of gable walls is similar to that of the roof itself, except for the trusses. The horizontal wind pressure on gable wall is resisted by a horizontal ceiling truss in the end bay, and it is equally distributed to each line of supporting posts by means of purlins or ceiling struts connecting the upper ends of the posts.

385. Programme.—Building open on all sides with closed gables; 100 × 160 ft. and 18 ft. high to roof; span of trusses 100 ft.; rise 20 ft.; 10 equal panels; cantilever of 20 ft. at each end, making columns 60 ft. on centres; covering of painted tin on longleaf pine sheathing; steel rafters, purlins, trusses, and columns; wind 30 lbs. per sq. ft. of vertical plane; location in 40° north latitude.

386. Dimensions.—

21.8° = inclination of roof.

10.77 ft. = inclined panel length.

10.77 × 16 = 172.32 sq. ft. = apex and purlin areas.

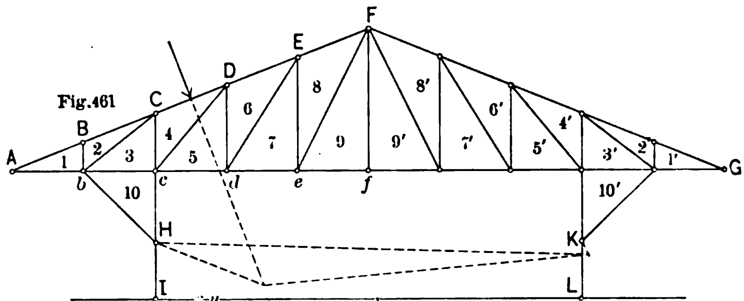


Fig. 461

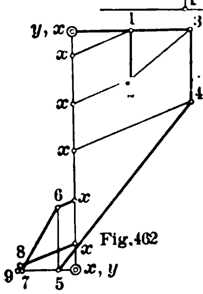


Fig. 462

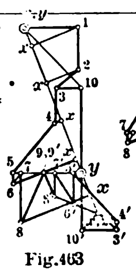


Fig. 463

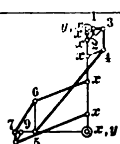


Fig. 464

387. Apex Loads.—

Truss = 4.794 lbs. per horizontal sq. ft. as before.

Snow = 2.5 (40° − 35°) = 12.5 lbs. per horizontal sq. ft.

Wind = 14.53 lbs. per inclined sq. ft. as before.

$P = 172.32\ (2 + 4 + 4 + 3 + 4.794 \cos 21.8°) = 3007$ lbs. $= 1.504$ tons.

$S = 172.32\ (12.5 \cos 21.8°) = 2001$ lbs. $= 1.001$ tons.

$W = 172.32 \times 14.53 = 2504$ lbs. $= 1.252$ tons.

388. Loads on Half Truss.—

Permanent $= 1.504 \times 5 = 7.52$ tons.

Snow $= 1.001 \times 5 = 5.01$ tons.

Wind $= 1.252 \times 5 = 6.26$ tons.

389. Stresses in Intermediate Truss.—Fig. 462 is the P stress diagram for the half truss. No stress here occurs in the diagonal connecting post and lower chord of truss, and the stress diagram is drawn as for Example 13.

The S stresses can be computed by the proportion:

1.504 : 1.001 : : P stress : S stress in same member.

Fig. 463 is the W stress diagram for the entire truss.

390. Stresses in Gable Truss.—Since this truss supports a half bay of the roof, the stresses in its members for this loading are just half those just found for the intermediate truss and thus may be computed from those.

The P loads of gable wall are supported at apexes of upper chord of this truss. These apex loads are as follows:

Apex.	Wall area.	Load.		
A	5 sq. ft.	5 × 13 #	= 65 #	= 0.033 ton.
B	40 sq. ft.	40 × 13 #	= 520 #	= 0.260 ton.
C	80 sq. ft.	80 × 13 #	= 1040 #	= 0.520 ton.
D	120 sq. ft.	120 × 13 #	= 1560 #	= 0.780 ton.
E	160 sq. ft.	160 × 13 #	= 2080 #	= 1.040 tons.
F	190 sq. ft.	190 × 13 #	= 2470 #	= 1.235 tons.

Fig. 464 is the P stress diagram for these loads on gable truss. These stresses are to be added to the half intermediate stresses to obtain the total P stress in each member, as shown in the stress sheet.

391. Horizontal Wind Loads on Gable Wall.—These must be supported by a horizontal ceiling truss in the end bay of the roof, to which they are transmitted from the verticals of the gable truss, whose lower ends are fastened to the ceiling truss, their upper ends being attached to the purlins of the roof, producing compression therein. A diagonal rod extends from apex of next intermediate truss to outer apex of ceiling truss in vertical plane through the purlin. Hence the horizontal component of the tension in this

rod = W load at upper end of vertical and is transmitted to apex of ceiling truss. Its V component produces a vertical load at apex of intermediate truss, which must be taken into account, as it slightly increases the stresses already found for that truss. These stresses and components are best determined graphically.

392. Table of Wind Loads on Gable.—

Vertical.	Wall area.	W load.		At each end of vert.
A	5 sq. ft.	5 × 30 # = 150 #	= 0.075 ton.	0.038 ton.
$B\ b$	40 sq. ft.	40 × 30 # = 1200 #	= 0.600 ton.	0.300 ton.
$C\ c$	80 sq. ft.	80 × 30 # = 2400 #	= 1.200 tons.	0.600 ton.
$D\ d$	120 sq. ft.	120 × 30 # = 3600 #	= 1.800 tons.	0.900 ton.
$E\ e$	160 sq. ft.	160 × 30 # = 4800 #	= 2.400 tons.	1.200 tons.
$F\ f$	190 sq. ft.	190 × 30 # = 5700 #	= 2.850 tons.	1.430 tons.

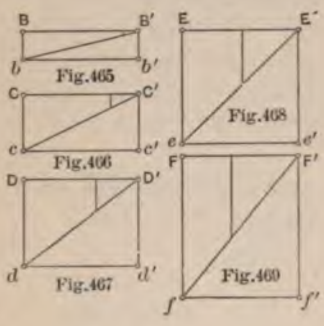


Fig. 465 is a vertical section through the verticals $B\ b$ and the two purlins, B and b, showing the diagonal rod $B'b$. Laying off 0.30 ton on $B'b$ and drawing a vertical we find 0.30 ton = compression in purlin, 0.33 ton = tension in rod, and 0.10 ton = load at apex B' of intermediate truss. Proceeding in like manner at the remaining apexes, the following results are obtained by Figs. 465 to 469.

393. Loads; Stresses in Purlins and Diagonal Rods.—

Apex.	H load.	Comp. in purlin.	Tension in rod.	V load on int. tr.
A	0.75 ton.	0.00 ton.	0.00 ton.	0.00 ton.
b	0.60 ton.	0.30 ton.	0.33 ton.	0.10 ton.
c	1.20 tons.	0.60 ton.	0.66 ton.	0.30 ton.
d	1.80 tons.	0.90 ton.	1.12 tons.	0.67 ton.
e	2.40 tons.	1.20 tons.	1.65 tons.	1.61 tons.
f	2.85 tons.	1.43 tons.	2.25 tons.	1.75 tons.

394. Stress Diagrams for Horizontal Truss.—Fig. 470 is the truss diagram for one-half the ceiling truss, the horizontal loads being applied at the lower apexes in the figure.

Fig. 471 is the corresponding W stress diagram.

395. Stress Diagram for Additional Loads on Second Truss.—Fig. 472 is the stress diagram for the additional loads on the next

intermediate truss, consisting of the V components of stresses in the diagonal rods, Figs. 465 to 469. These are to be added to the stresses already found for intermediate truss, as in the following stress sheet, but omitting the W stresses from wind pressure on roof, since the wind cannot act on roof and gable at the same time with the maximum pressure.

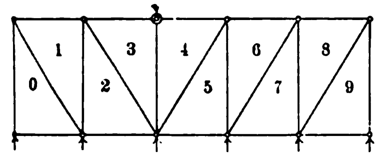


Fig. 470.

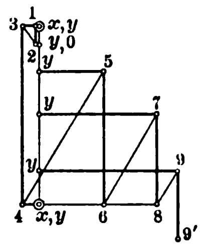


Fig. 471.

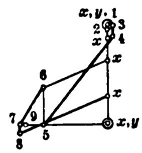


Fig. 472.

396. Stress Sheet for Intermediate Truss.—

Member.	P-stress.	S-stress.	W-stress W.	W-stress L.	Maximum.	Minimum.
X 1	+ 2.0	+ 1.3	+ 1.7	+ 0.0	+ 5.0	+ 2.0
X 2	+ 2.0	+ 1.3	+ 1.2	+ 0.0	+ 4.5	+ 2.0
X 4	+ 4.1	+ 2.7	− 0.1	+ 0.7	7.5	+ 4.0
X 6	− 0.6	− 0.4	− 2.0	− 0.7	− 3.0	− 0.6
X 8	− 2.0	− 1.3	− 2.4	− 1.4	− 5.7	− 2.0
Y 1	− 1.9	− 1.3	− 1.8	− 0.0	− 5.0	− 1.9
Y 3	− 3.8	− 2.5	+ 0.7	− 1.2	− 7.5	− 3.8
Y 5	+ 0.6	+ 0.4	+ 2.1	+ 0.2	+ 3.1	+ 0.6
Y 7	+ 1.9	+ 1.3	+ 1.9	+ 0.8	+ 5.1	− 1.9
Y 9	+ 1.9	+ 1.3	+ 1.2	+ 1.2	+ 4.4	+ 1.9
1 2	− 1.5	− 1.0	− 1.4	− 0.0	− 3.9	+ 1.5
3 4	− 2.2	− 1.5	− 1.1	− 0.3	− 4.8	− 2.2
5 6	+ 2.0	+ 1.3	− 0.3	+ 1.0	+ 4.3	+ 2.0
7 8	0.0	0.0	− 1.5	+ 0.7	− 1.5	+ 0.7
9 9′	0.0	0.0	0.0	0.0	0.0	0.0
2 3	+ 2.4	+ 1.6	− 0.8	+ 0.8	+ 4.8	+ 1.6
4 5	− 6.9	− 4.6	− 2.2	− 2.0	−13.7	− 6.9
6 7	− 2.4	− 1.6	+ 0.3	− 1.2	− 5.2	− 2.1
8 9	0.0	0.0	+ 1.7	− 0.9	+ 1.7	− 0.9
Diag. 10	0.0	0.0	− 2.6	+ 0.7	− 2.6	+ 0.7
Post 10	− 7.5	− 5.0	− 2.7	− 1.8	−15.2	− 7.5
3 10	− 3.8	− 2.5	+ 0.7	− 1.2	− 7.5	− 3.1

397. Stress Sheet for Intermediate Truss next Gable Truss.—

Member.	*P*-stress.	*S*-stress.	*G*-stress.	Maximum.	Minimum.
X 1	+ 2.0	+ 1.3	+ 0.0	+ 3.3	+ 2.0
X 2	+ 2.0	+ 1.3	+ 0.0	+ 3.3	+ 2.0
X 4	+ 4.1	+ 2.7	+ 0.2	+ 7.0	+ 4.1
X 6	− 0.6	− 0.4	− 2.3	− 3.3	− 0.6
X 8	− 2.0	− 1.3	− 3.1	− 6.4	− 2.0
Y 1	− 1.9	− 1.3	− 0.0	− 3.2	− 1.9
Y 3	− 3.8	− 2.5	− 0.2	− 6.5	− 3.8
Y 5	+ 0.6	+ 0.4	+ 2.2	+ 3.2	− 0.6
Y 7	+ 1.9	+ 1.3	+ 2.9	+ 6.1	+ 1.9
Y 9	+ 1.9	+ 1.3	+ 2.7	+ 5.9	+ 1.9
1 2	− 1.5	− 1.0	− 0.1	− 2.6	− 1.5
3 4	− 2.2	− 2.2	− 0.4	− 4.8	− 2.2
5 6	+ 2.0	+ 1.3	+ 1.2	+ 4.5	+ 2.0
7 8	0.0	0.0	− 0.3	− 0.3	0.0
9 9′	0.0	0.0	0.0	0.0	0.0
2 3	+ 2.4	+ 1.6	+ 0.2	+ 4.2	+ 2.4
4 5	− 6.9	− 4.6	− 3.7	−15.2	− 6.9
6 7	− 2.4	− 1.6	− 1.4	− 5.4	− 2.4
8 9	0.0	0.0	+ 0.3	+ 0.3	0.0
Diag. 10	0.0	0.0	0.0	0.0	0.0
Post 10	− 7.5	− 5.0	0.0	−12.5	− 7.5
3 10	− 3.8	− 2.5	− 0.2	− 6.5	− 3.8

398. Stress Sheet for Gable Truss.—

Member.	*P*-stress.	*S*-stress.	*W*-stress *W*.	*W*-stress *L*.	*P*-gable.	Maximum.	Minimum.
X 1	+ 1.0	+ 0.7	+ 0.9	+ 0.0	+ 0.2	+ 2.7	+ 1.2
X 2	+ 1.0	+ 0.7	+ 0.6	+ 0.0	+ 0.2	+ 2.5	+ 1.2
X 4	+ 2.1	+ 1.4	− 0.1	+ 0.4	+ 0.6	+ 4.5	+ 2.6
X 6	− 0.3	− 0.2	− 1.0	− 0.4	− 1.8	− 3.3	− 2.1
X 8	− 1.0	− 0.7	− 1.2	− 0.7	− 2.5	− 5.4	− 3.5
Y 1	− 1.0	− 0.7	− 0.9	0.0	− 0.2	− 2.8	− 1.2
Y 3	− 1.9	− 1.3	+ 0.4	− 0.6	− 0.5	− 4.3	− 2.0
Y 5	+ 0.3	+ 0.2	+ 1.1	+ 0.1	+ 1.7	+ 3.3	+ 2.0
Y 7	+ 1.0	+ 0.7	+ 1.0	+ 0.4	+ 2.3	+ 5.0	+ 3.3
Y 9	+ 1.0	+ 0.7	+ 0.6	+ 0.6	+ 2.1	+ 4.4	+ 3.1
1 2	− 0.8	− 0.5	− 0.7	0.0	− 0.3	− 2.3	− 1.1

Member.	*P*-stress.	*S*-stress.	*W*-stress *W*	*W*-stress *L*.	*P*-gable.	Maximum.	Minimum.
3 4	− 1.1	− 1.1	− 0.6	− 0.2	− 0.7	− 3.5	− 1.8
5 6	+ 1.0	+ 0.7	− 0.2	+ 0.5	+ 1.0	+ 3.2	− 1.5
7 8	0.0	0.0	− 0.8	+ 0.4	− 0.3	− 1.1	+ 0.1
9 9′	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2 3	+ 1.2	+ 0.8	− 0.4	+ 0.4	+ 0.4	+ 2.8	+ 1.2
4 5	− 3.5	− 2.3	− 1.1	− 1.0	− 3.4	−10.3	− 6.9
6 7	− 1.2	− 0.8	+ 0.2	− 0.6	− 1.2	− 3.8	− 2.4
8 9	0.0	0.0	+ 0.9	− 0.5	+ 0.3	+ 1.2	− 0.2
Diag. 10	0.0	0.0	− 1.3	+ 0.4	0.0	− 1.3	+ 0.4
Post 10	− 3.8	− 2.5	− 1.4	− 0.9	− 3.3	−11.0	− 7.1
3 10	− 1.9	− 1.3	+ 0.4	− 0.6	0.0	− 3.8	− 1.5

399. Stress Sheet for Ceiling Truss, Purlins and Diagonal Rods.—

Member.	*W*-stress.	Purlin.	Diagonal rod.
X 2	+ 0.2	*A B′* − 0.3	*B′ b* + 0.3
X 5	− 3.5	*C C′* − 0.6	*C′ c* + 0.7
X 7	− 5.8	*D D′* − 0.9	*D′ d* + 1.1
X 9	− 6.7	*E E′* − 1.2	*E′ e* + 1.7
Y 1	− 0.2	*F F′* − 1.4	*F′ f* + 2.3
Y 3	− 0.6	....	....
Y 4	0.0	....	....
Y 6	+ 3.5	....	....
Y 8	+ 5.9	....	....
1 2	− 0.7	....	....
3 4	0.0	....	....
5 6	− 5.6	....	....
7 8	− 3.8	....	....
9 9′	− 2.9	....	....
0 1	+ 0.3	....	....
2 3	+ 1.0	....	....
4 5	+ 6.6	....	....
6 7	+ 4.5	....	....
8 9	+ 1.7	....	....

400. Loads Transmitted to Posts.—The total horizontal force transmitted to posts supporting each end of the ceiling truss $= 0.75 + 0.6 + 1.2 + 1.8 + 2.4 + \frac{2.85}{2} = 8.18$ tons. Then the horizontal

force acting at the top of each side post from the end of the building $= \dfrac{8.18}{11} = 0.74$ ton.

The W resultant for intermediate truss at H on the windward post = 4.90 tons. In Fig. 473, this is resolved into a V component of 4.55 tons and an H component of 1.80 tons. Similarly the W resultant at K on leeward post = 1.36 tons, which may be resolved into a V component of 1.26 tons and an H component of 0.50 ton.

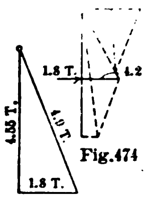


Fig. 474
Fig. 473

The windward post is subject to a maximum bending moment at H, there resisting an H force of 1.80 tons, which produces a transverse bending moment of $2 \times 4.2 = 8.4$ foot-tons, by Fig. 474. Of this H component, 0.80 ton is resisted at top and 1.00 ton at bottom of the post.

401. Foundations of Posts.—These are assumed to be massive reinforced concrete blocks 2 ft. deep, to which the lower ends of the posts are firmly attached, but not anchored.

402. Foundation for Second Post from Angle.—For windward footing of post supporting intermediate truss next gable truss.

Maximum V load $= 7.52 + 5.01 + 4.55 + 3.14 = 20.22$ tons.

Maximum H component at $H = 1.80$ tons.

Assume 1.50 tons per sq. ft. as the maximum safe compression on the soil beneath.

Assume a footing $5.5 \times 3.5 + 2.0$ ft.; 19.25 sq. ft. area; 38.5 cu. ft. volume; 2.89 tons weight.

Total V component $= 20.22 + 2.89 = 23.11$ tons.

The V and H components here act at the top and middle of length of the footing. Hence determine their resultant in Fig. 475, an eccentricity of 0.15 ft. between middle vertical and resultant at bottom of block. Average intensity of pressure on soil $= \dfrac{23.11}{19.25} = 1.20$ tons per sq. ft. Maximum intensity of pressure $= 1.20\left(1 + \dfrac{6 \times 0.15}{5.5}\right) = 1.40$ tons per sq. ft. These pressures are laid off in Fig. 475 for comparison and are safe.

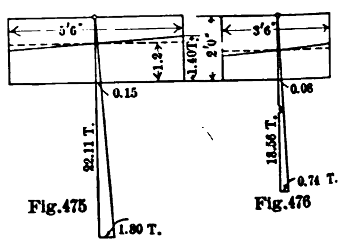


Fig. 475
Fig. 476

When the wind acts on the end of the building instead of its roof, the V load on same footing $= 7.52 + 5.01 + 3.14 + 2.89$ = a total of 18.56 tons. The H component then = 0.74 ton. Laying these off in Fig. 476, the eccentricity at bottom of footing = 0.08 ft.

$$\text{Average intensity of pressure} = \frac{18.56}{19.25} = 0.966 \text{ ton per sq. ft.}$$

$$\text{Maximum intensity of pressure} = 0.966\left(1 + \frac{6 \times 0.08}{3.5}\right) = 1.10 \text{ tons}$$

per sq. ft. Therefore this footing is entirely safe for the intermediate columns with the greatest loads. The footing for the other intermediate columns may be a little shorter.

403. Foundation for Other Intermediate Posts.—Maximum V load on footing of same $= 20.22 - 3.14 = 17.08$ tons.

Assume footing $5 \times 3.5 \times 2$ ft. Area = 17.5 sq. ft.; volume = 35 cu. ft.; weight = 5250 lbs. = 2.63 tons.

Total V component $= 17.08 + 2.63 = 19.71$ tons. H component = 1.80 tons. By Fig. 477, eccentricity = 0.18.

Average intensity of pressure = $\dfrac{19.71}{17.5} = 1.126$ tons per sq. ft.

Maximum intensity of pressure $= 1.126\left(1 + \dfrac{6 \times 0.18}{5.0}\right) = 1.369$ tons per sq. ft.

Fig. 477

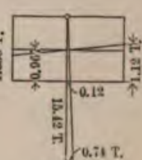


Fig. 478

When the wind acts on the end only of the building, V component $= 7.52 + 5.01 + 2.63 = 15.16$ tons. H component = 0.74 ton as before.

By Fig. 478, eccentricity = 0.12 ft.

$$\text{Average intensity of pressure} = \frac{15.16}{17.50} = 0.967 \text{ ton per sq. ft.}$$

$$\text{Maximum intensity of pressure} = 0.967\left(1 + \frac{6 \times 0.12}{3.5}\right) = 1.117$$

tons per sq. ft. Hence these footings for intermediate columns are entirely safe.

404. Foundation for Angle Post.—Corner column on windward side of building.

$$V \text{ load} = \frac{7.52 + 5.01 + 4.55}{2} + 3.25 = 11.79 \text{ tons.}$$

longleaf pine sheathing, steel rafters, purlins, and trusses; trusses 100 ft. span, 20 ft. rise, and 16 ft. on centres; reënforced concrete footings for steel posts, which are braced to lower chord inside and to steel plate sidewise at mid-height A; end posts support ends of

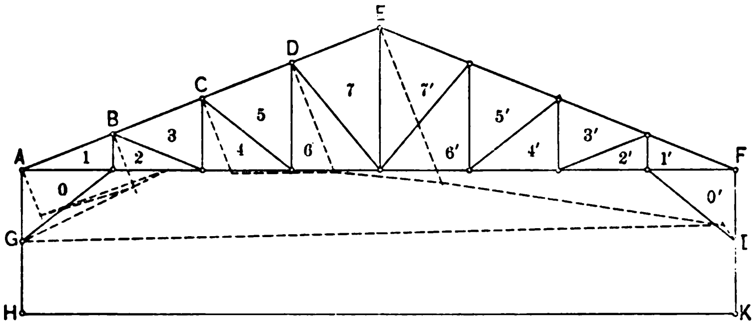


Fig. 481.

purlins at top and are attached to ceiling truss at crossing; wind pressure acting on post is entirely supported by ceiling truss and footing; wind pressure 30 lbs. per vertical sq. ft.; location at Baltimore, Md.

407. Dimensions.—21.8° = inclination of roof.

$\sqrt{12.5^2 + 5^2} = \sqrt{181.25} = 13.46$ = inclined panel length.

$13.46 \times 16.00 = 215.36$ = apex and purlin area.

$20 \times 16 = 320$ sq. ft. = area of one bay of side wall.

408. Apex Loads.—

Truss = 4.794 lbs. per horizontal sq. ft. as before.

Snow = 2.5 (40° − 35°) = 10 lbs. per horizontal sq. ft.

Wind = 14.53 lbs. per inclined sq. ft. of roof.

Wind = 30 lbs. per vertical sq. ft. of wall.

$P = 215.36\,(6+4+4+3+4.794 \cos 21.8°) = 4619$ lbs. $= 2.310$ tons.

$S = 215.36\,(10 \cos 21.8°) = 2085$ lbs. $= 1.043$ tons.

$W = 215.36 \times 14.53 = 3129$ lbs. $= 1.565$ tons.

W (for wall) $= 320 \times 30 = 9600$ lbs. $= 4.800$ tons.

409. Loads on Half Truss.—

Permanent = $2.310 \times 3.5 = 8.09$ tons.

Snow = $1.043 \times 3.5 = 3.65$ tons.

Wind = $1.565 \times 4.0 = 6.26$ tons for roof.

Wind = $4.800 \times 0.5 = 2.40$ tons at top of column.

410. Stress Diagrams for Intermediate Truss.—Fig. 484 is the P stress diagram for an intermediate truss. Snow stresses may be found by the proportion: 2.310 : 1.043 : : P stress : S stress in the same member.

Fig. 485 is the W stress diagram for the same truss, but it is drawn at twice the scale of Fig. 484 to clearly show the stresses in the leeward members.

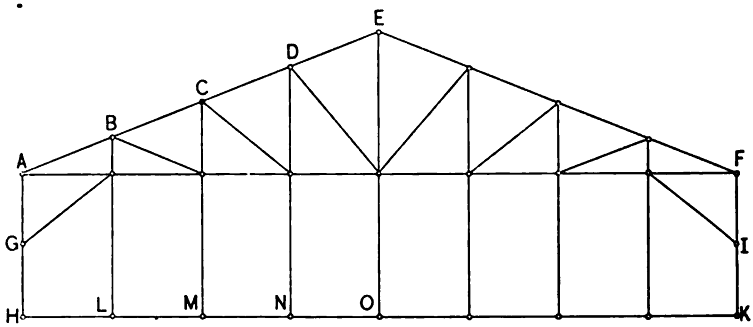


Fig. 482

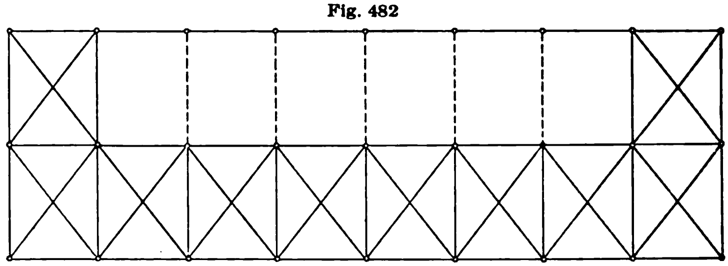


Fig. 483

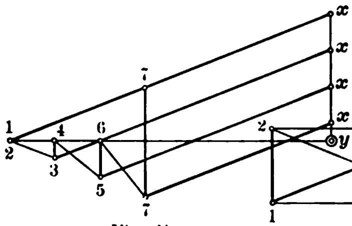


Fig. 484

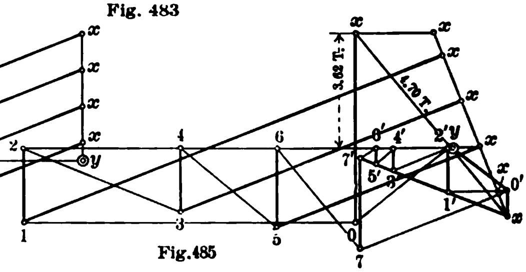


Fig. 485

411. Stresses in Gable Truss.—For the gable end truss, the P and S loads at B, C, D, E, etc., can rest directly on the tops of the end posts, then being directly transferred to the footings; the W loads alone are supported by the truss and are transmitted to the corner posts by the truss. The W stresses in members of the gable

truss are just half those for the intermediate truss. The P weights of the walls are directly supported by the posts and their footings.

412. Loads on Gable Posts.—

Post.	Wall area.	P load.	W load.
A H	132.8 sq. ft.	0.863 ton.	1.992 tons.
B L	312.5 "	2.031 tons.	4.688 "
C M	375.0 "	2.438 "	5.625 "
D N	437.5 "	2.844 "	6.563 "
E O	471.3 "	3.064 "	7.070 "
Int.	320.0 "	2.080 "	4.800 "

The P weight of the wall is taken at 13 lbs. per sq. ft. of vertical area. The centre of the wind pressure on the portion of the wall supported by a post may be taken approximately at its mid-height, as indicated in Fig. 482, except for the post *E O*. The upper end of the post may be supported against the horizontal wind pressure in either of two ways.

413. Support of Loads on Post.—1. As in Example 8, the upper end of the post joins the purlin, a rod extending from the corresponding apex of the next truss diagonally in the end bay to the outer edge of the ceiling truss in end bay. Then half the W load on the post is resisted by the ceiling truss and half directly by the footing, except for middle post *E O*, where less than half is resisted by the ceiling truss at *e*.

414. Connection of Post and Ceiling Truss.—2. The posts may be directly attached to the ceiling truss at its outer edge, the upper part of the post then forming a cantilever. This shows a much larger proportion of the W loads on the ceiling truss and eventually on the side posts, but it omits the compression in the purlins and also the diagonal rods, so that it is probably just as economical in practice. For sake of variety, the second method will be applied in this example. The W load on a gable end post is divided between the ceiling truss and the footing inversely as the centre of gravity of the W pressure area on post divides the clear height of the post, which is here uniformly 20 ft.

415. Distribution of Wind Loads on Posts.—

Post.	Total W load.	At ceiling truss.	At footing.
A H	1.992 tons.	0.996 ton.	0.996 ton.
B L	4.688 "	2.953 tons.	1.735 tons.
C M	5.625 "	4.275 "	1.350 "
D N	6.563 "	5.743 "	0.820 ton.
E O	7.070 "	6.646 "	0.424 "

416. Stress Diagram for Ceiling Truss.—This arrangement produces a W load of 17.29 tons on one-half the ceiling truss, or $\frac{17.29}{11} = 1.572$ tons on each side post at its top.

Fig. 486 is the half diagram of the ceiling truss in end bay.

Fig. 487 is half the wind stress diagram for the same.

417. Stress Sheet for Intermediate Truss.—

Member.	P-stress.	S-stress.	W-stress W.	W-stress L.	Maximum.	Minimum.
X 1	−21.7	− 9.8	−14.2	− 2.0	−45.7	−21.7
X 3	−18.8	− 8.5	− 9.5	− 3.9	−36.8	−18.8
X 5	−15.8	− 7.1	− 6.8	− 4.5	−29.7	−15.8
X 7	−12.6	− 5.7	− 4.7	− 5.0	−23.3	−12.6
Y 1	+20.2	+ 9.1	+10.5	+ 1.9	+39.8	+20.2
Y 2	+20.2	+ 9.1	+13.5	+ 0.1	+42.8	+20.2
Y 4	+17.4	+ 7.9	+ 8.5	+ 1.9	+33.8	+17.4
Y 6	+14.6	+ 6.6	+ 5.5	+ 2.4	+26.7	+14.6
1 2	0.0	0.0	+ 2.4	+ 1.4	+ 2.4	+ 1.4
3 4	+ 1.1	+ 0.5	+ 2.0	+ 0.7	+ 3.6	+ 1.1
5 6	+ 2.3	+ 1.0	+ 2.5	+ 0.5	+ 5.8	+ 2.2
7 7′	+ 7.0	+ 3.2	+ 2.9	+ 2.9	+13.1	+ 7.0
2 3	− 3.0	− 1.4	− 5.4	− 2.0	− 9.8	− 3.0
4 5	− 3.6	− 1.6	− 3.9	− 0.7	− 9.1	− 3.6
6 7	− 4.6	− 2.1	− 4.1	− 0.6	−10.8	− 4.6
Post 0	0.0	0.0	− 6.0	− 0.8	− 6.0	− 0.8
Diag. 0	0.0	0.0	+ 3.8	+ 2.3	+ 3.8	+ 3.8

418. Stress Sheet for Ceiling Truss.—

Member.	W stress.	Member.	W stress.	Member.	W stress.
X 2	− 12.7	Y 7	+ 32.7	2 3	+ 16.8
X 4	− 23.0	1 2	− 16.2	4 5	+ 11.5
X 6	− 30.2	3 4	− 13.2	6 7	+ 4.1
Y 1	+ 12.7	5 6	− 9.0		
Y 3	+ 23.0	7 7′	− 6.7		
Y 5	+ 30.2	X 1	+ 20.7		

419. Bending Moments on Posts.—According to Fig. 485, the inclined wind reaction at $G = 4.70$ tons. Its V component $= 3.62$ tons and its H component $= 3.00$ tons, which produces the maximum bending moment acting on the side post from the side. Hence

$$M_{max} \text{ at } G = \frac{3.00 \times 20}{4} = 15.00 \text{ ft.-tons.}$$

Also for the corner post, M_{max} at $G = \dfrac{1.50 \times 20}{4} = 7.50$ ft.-tons sidewise; M_{max} at $G = \dfrac{1.992 \times 20}{4} = 9.96$ ft.-tons endwise the building.

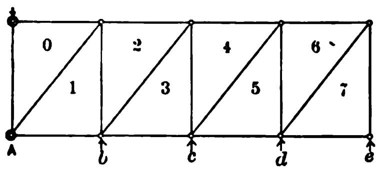


Fig. 486.

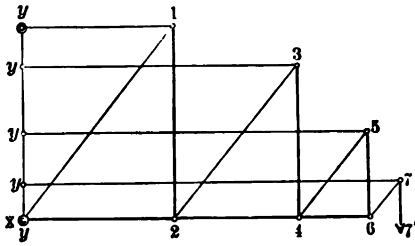


Fig. 487.

420. Middle End Post E O (Fig. 488).—The vertical $E\,O$ represents the axis of the post, the portion $E\,o$ not being loaded by W pressure. $O\,e = 20$ ft. The post is supported at the footing O and at e, the outer chord of the ceiling truss. Hence the part $E\,e$ is a cantilever post. The height of the post between floor and

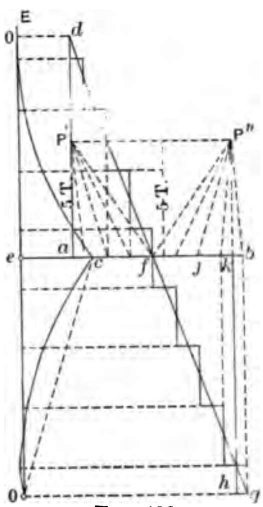


Fig. 488.

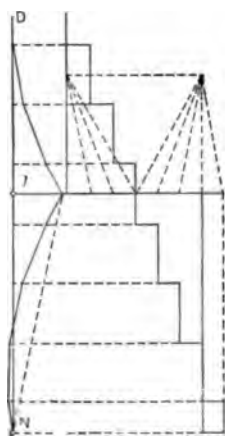


Fig. 489.

ceiling is divided into 4 equal parts of 5 ft. each in height, on which the W load $= 0.938$ ton each. There are 3 similar divisions and loads above the ceiling truss, together with a fractional portion of 4 ft. and 0.750 ton at top. Each load is here assumed to act horizontally

through its mid-height. The load line $a\ b$ is drawn horizontally through e and the successive loads are laid off from a to b. Drawing the vertical $a\ P' = 5.00$ tons, for example, P' is the pole for the force polygon for the cantilever $E\ e$, for which the equilibrium polygon $o\ c$ and the stepped shear line $i\ f$ are then drawn, beginning at o and locating the point c on the horizontal $e\ b$. Join $E\ c$ and through the middle point i of the load line $f\ b$, for the portion $e\ O$ of the post, draw $i\ P''$ parallel to $O\ c$. The equilibrium polygon $O\ c$ and stepped shear line $f\ g$ are then drawn. Drop the vertical $P''\ h$ through P'' and it becomes the shear axis, $f\ k$ being the shear = reaction at e, and $h\ g = H$ at O. The maximum shear and maximum bending moment evidently both occur at e, the latter = pole distance $P'\ a \times$ intercept $e\ c = 5.00$ T. $\times$ 6.00 ft. = 30 ft.-tons. Maximum shear = 3.45 tons. The true equilibrium curve for the uniform load may be drawn tangent at the middle point of each division and the straight shear line as in Fig. 488, but these do not change the values of maximum moment and shear at e, and this is then unnecessary.

421. Post D N.—The same method is then applied to the post $D\ N$ in Fig. 489, obtaining maximum shear at $d = 2.80$ tons and maximum bending moment there $= 5.00 \times 4.1$ ft. = 20.50 ft.-tons.

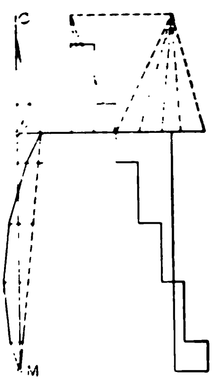


FIG. 490.

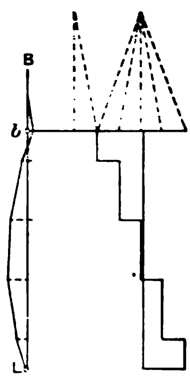


FIG. 491.

422. Post C M.—For post $C\ M$ in Fig. 490, maximum shear = 2.40 tons and maximum bending moment $= 5.00 \times 1.9$ ft. = 9.50 ft.-tons, both at c.

423. Post B L.—For post $B\ L$ in Fig. 491, maximum shear = 1.96 tons and maximum bending moment = $5.00 \times 0.5 = 2.50$ ft.-tons, both at B.

424. Reinforced Concrete Footings of Posts.—

425. Intermediate Side Post.—

Vertical load = $9.24 + 4.17 + 2.08 + 3.62 = 19.11$ tons from side.

H component from side = $3.00 + 4.80 = 7.80$ tons.

H component from end = 1.57 tons.

Assume a footing block $8 \times 3 \times 2$ ft. deep; area = 24 sq. ft.; volume = 48 cu. ft.; weight = 7200 lbs. = 3.60 tons.

Total V component = $19.11 + 3.60 = 22.71$ tons from side.

H component = 7.80 tons from side.

By Fig. 492, eccentricity of resultant = 0.68 ft.

Average intensity of pressure on soil $= \dfrac{22.71}{24} = 0.948$ ton per sq. ft.

Maximum intensity of pressure $= 0.948\left(1 + \dfrac{6 \times 0.68}{8}\right) = 1.43$ tons per sq. ft.

Total V component = $22.71 - 3.62 = 19.09$ tons from end.

H component = 1.57 tons from end.

By Fig. 493, eccentricity of resultant = 0.16 ft.

Average intensity of pressure on soil $= \dfrac{19.09}{24} = 0.796$ ton per sq. ft.

Maximum intensity of pressure $= 0.796\left(1 + \dfrac{6 \times 0.16}{3}\right) = 1.05$ tons per sq. ft.

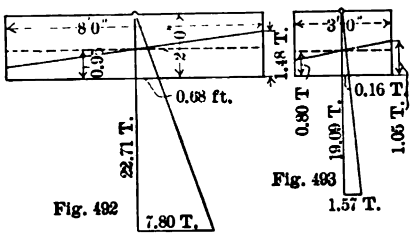


Fig. 492

Fig. 493

Neglecting resistance to sliding on the soil beneath, the resistance of soil to sliding of the block endwise by the horizontal component = $6 \times 1.50 = 9.00$ tons, while the H component only = 7.80 tons.

Therefore this footing block of reinforced concrete is safe for the intermediate columns.

426. Footings of Angle Post (Fig. 482).—

$$V \text{ load from side only} = \frac{19.11}{2} = 9.56 \text{ tons.}$$

$$V \text{ load from end only} = \underline{0.86} \text{ ton.}$$

$$\text{Total } V \text{ load} = 10.42 \text{ tons.}$$

Assume footing 5 1 2 × 3 × 2 ft.; area = 16.5 sq. ft.; volume = 33 cu. ft.; weight = 2.63 tons.

V component = 10.42 + 2.63 = 13.05 tons from side.

$$H \text{ component} = \frac{7.80}{2} = 3.90 \text{ tons from side.}$$

By Fig. 494, eccentricity of resultant = 0.60 ft.

$$\text{Average intensity of pressure} = \frac{13.05}{17.5} = 0.746 \text{ ton per sq. ft.}$$

from side.

$$\text{Maximum intensity of pressure on soil} = 0.746\left(1 + \frac{6 \times 0.60}{5.5}\right) =$$

1.234 T.

V component = 8.61 + 2.63 = 11.24 tons from end.

H component = 1.00 ton from end.

$$\text{Average intensity of pressure} = \frac{11.24}{17.5} = 0.643 \text{ ton per sq. ft.}$$

from end.

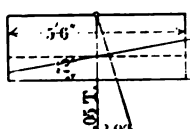


Fig. 494

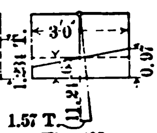


Fig. 495

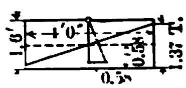


FIG. 496.

By Fig. 495, eccentricity of resultant = 0.20 ft.

$$\text{Maximum intensity of pressure} = 0.643\left(1 + \frac{6 \times 0.20}{3}\right) = 0.90 \text{ T.}$$

per sq. ft.

427. Post B L (Fig. 482).—

Assume footing 4 × 1.5 × 1.5 ft.; area = 6.00 sq. ft.: volume = 9.00 cu. ft.; weight = 0.675 ton.

V component $= \dfrac{2.310}{2} + \dfrac{1.043}{2} + 2.031 + 0.675 = 4.39$ ns.

H component $= 1.74$ tons.

By Fig. 496, eccentricity of resultant $= 0.58$ ft.

Average intensity $= \dfrac{4.39}{6.00} = 0.732$ ton.

Maximum intensity $= 0.732\left(1 + \dfrac{6 \times 0.58}{4}\right) = 1.368$ tons per . ft.

428. Post C M (Fig. 482).—

Assume footing $4 \times 1.5 \times 1.5$ as before; area $= 6.00$ sq. ft.; lume $= 9.00$ cu. ft.; weight $= 0.675$ ton.

V component $= \dfrac{2.310}{2} + \dfrac{1.043}{2} + 2.438 + 0.675 = 4.79$ tons.

H component $= 1.35$ tons.

By Fig. 497, eccentricity $= 0.45$ ft.

Average intensity $= \dfrac{4.79}{6.00} = 0.798$ tons per sq. ft.

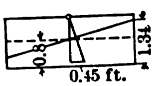


FIG. 497.

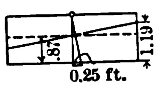


FIG. 498.

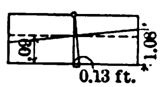


FIG. 499

Maximum intensity $= 0.798\left(1 + \dfrac{6 \times 0.45}{4}\right) = 1.34$ tons per sq. ft.

429. Post D N (Fig. 482).—

V component $= \dfrac{2.310}{2} + \dfrac{1.043}{2} + 2.844 + 0.675 = 5.20$ tons.

By Fig. 498, eccentricity $= 0.25$ ft.

Average intensity $\dfrac{5.20}{6.00} = 0.867$ ton per sq. ft.

Maximum intensity $= 0.867\left(1 + \dfrac{6 \times 0.25}{4}\right) = 1.191$ tons per sq. ft.

430. Post E O (Fig. 482).—

V component $= \dfrac{2.310}{2} + \dfrac{1.043}{2} + 3.064 + 0.675 = 5.42$ tons.

15

By Fig. 499, eccentricity = 0.13 ft.

$$\text{Average intensity} = \frac{5.42}{6.00} = 0.904 \text{ tons per sq. ft.}$$

$$\text{Maximum intensity} = 0.904\left(1 + \frac{6 \times 0.13}{4}\right) = 1.084 \text{ tons per sq. ft.}$$

The examples studied in this chapter show clearly the necessity for carefully determining the stability of the entire structure, as well as that of the roof.

CHAPTER IX

SAFE STRENGTH OF MATERIALS

431. Simplification of Formulas, Data, and Computations.—Long experience in the study and application of the formulas and data comprised in this chapter has clearly shown:

1. That the usual formulas for strength of materials may be put into simpler and more convenient forms for practical use.

2. That the necessary computations are greatly abbreviated by some changes in notation and coefficients.

These improvements consist in taking the loads and coefficients of strength in net tons instead of in lbs.; bending moments in foot-tons instead of inch-lbs.; lengths in feet instead of inches, and distances between centres of beams and purlins in feet instead of inches. Sectional dimensions and distances between centres of rafters and joists are always taken in inches.

432. Notation Employed.—

Let W = load in tons producing the stress.
A = effective area of cross-section in square inches.
P = stress in tons per square inch of cross-section.
L = length of member or beam in feet.
b = breadth of a rectangular cross-section in inches.
d = depth of a rectangular cross-section in inches.
d = diameter of circular cross-section in inches.
I = moment of inertia of cross-section about a horizontal gravity axis.
c = distance in inches from this axis to most distant fibre of section.
c' = same distance to top fibre.
c'' = same distance to bottom fibre.
$\frac{I}{c}$ = section modulus of cross-section.
R_o = radius of gyration for any axis of section.
R_1 = radius of gyration for a horizontal axis.
R_2 = radius of gyration for a vertical axis.

T = maximum safe tension in tons per square inch.
S = maximum safe cross shear in tons per square inch.
S' = maximum safe shear with fibres in tons per square inch.
C = maximum safe compression in tons per square inch.
C' = maximum safe compression across fibres in tons per square inch.
E = modulus of elasticity in tons.
F = modulus of rupture in tons per square inch.
Δ = actual maximum deflection in inches.
t = thickness of wooden sheathing or flooring.
e = distance in inches between centres of rafters or joists.
w = load in lbs. per square foot on roofs, floors, etc.
M = maximum bending moment in foot-tons acting at any point.

Additional notations will be used when required for special formulas.

433. Values of Coefficients of Strength.—The values of the coefficients T, S, S', C, C', E, and F given in the following table are fair averages based on results of numberless experiments made by many investigators and on the requirements of building ordinances of many cities. They apply to materials of good quality, such as would be passed by any competent inspector or superintendent. It is impossible to give a series of coefficients satisfying all building ordinances, since these vary greatly from each other.

434. Neglect of Deflection.—It is rather singular that no city building ordinance, known to the author, requires consideration of the deflection of a beam or girder, although a long beam may safely resist breaking and yet may deflect enough to become unsightly and to crack any plastering supported by it. Therefore in the formulas here given for safe resistance to bending, the maximum permissible deflection in inches is taken at 3/100 of the length in feet, a value commonly employed in practice (40).

435. Table of Coefficients for Materials.—

Material.	T.	S.	S'.	C.	C'.	E.	F.
Birch	.50	.40	.10	.60	.20	700.	.55
Cedar	.40	.30	.05	.55	.10	450.	.45
Chestnut	.45	.35	.07	.60	.125	500.	.50
Cypress	.50	.40	.05	.55	.10	550.	.50
Elm	.50	.40	.10	.60	.15	600.	.55

Material.	T.	S.	S'.	C.	C'.	E.	F.
ıington........	.65	.50	.065	.80	.15	700.	.70
....	.50	.40	.04	.55	.15	650.	.55
....	.40	.30	.04	.55	.10	450.	.45
;.............	1.50	1.25	....	5.00	....	8000.	3.00 comp. 1.50 tens.
ught..........	6.00	4.50	....	6.00	....	14000.	6.00
....	.40	.30	.06	.60	.125	650.	.50
verage.........	.45	.40	.08	.70	.20	650.	.55
ıgar...........	.65	.50	.125	.85	.30	800.	.75
rage...........	.50	.45	.10	.60	.25	700.	.55
....	.45	.35	.08	.60	.20	650.	.50
te............	.60	.50	.125	.75	.30	750.	.65
gleaf..........	.65	.50	.075	.70	.175	850.	.70
rway..........	.40	.30	.05	.60	.10	600.	.50
:h............	.45	.35	.06	.65	.125	600.	.55
rtleaf.........	.50	.40	.04	.60	.125	600.	.55
ite............	.40	.30	.04	.50	.10	500.	.45
ellow.........	.30	.25	.04	.50	.075	500.	.45
....	.35	.30	.05	.50	.075	350.	.40
....	.50	.40	.06	.60	.125	650.	.55
ıt............	8.00	6.00	....	8.00	....	15000.	8.00
led............	8.00	6.00	....	8.00	....	14500.	8.00

Tension.—For simple tension, the resultant of the longitu-ress in a member is assumed to coincide with its longitudinal axis. Hence the tensile stress has a uniform intensity over ire effective cross-section. Holes for rivets, adding $\frac{1}{8}$ inch eter of rivet to allow for injury to metal by punching, must ıcted from cross-section to obtain net or effective area of

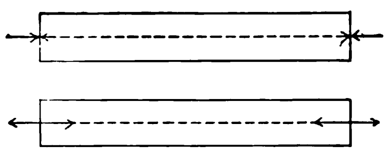


FIG. 500.

n $W = A\,T$ = maximum safe resistance in tons to ten- . . . . (41)

; = minimum safe net area in sq. in. for section. . . . (42)

$= \frac{\pi\, d^2}{4}$ for round rods,

$$W = \frac{\pi d^2 T}{4} = 0.787\ d^2\ T = \text{maximum safe resistance in tons.} \quad (43)$$

$$d = \sqrt{\frac{4 W}{\pi T}} = 1.128 \sqrt{d^2 T} = \text{minimum safe diameter in inches.} \quad (44)$$

$$d = 0.4\sqrt{W} = \text{minimum safe diameter of a steel rod.} \quad . \quad (45)$$

The last two formulas are only applicable to steel rods with upset or enlarged ends for nuts, since otherwise a deduction must be made for loss of sectional area and strength by cutting screw threads on end.

437. Shear, Transverse.—Transverse shear occurs in all materials; longitudinal shear only in wooden timbers parallel to the fibres. Both are assumed to be uniformly distributed over the entire section exposed to shearing. But this is not actually the case, for on the cross-section of a rectangular beam, the maximum intensity of the transverse shear is found at the middle of its depth and = $1\frac{1}{2}$ times the average intensity.

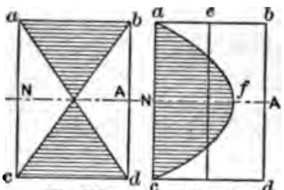


Fig.501 Fig.502

Fig. 501 illustrates the distribution of the longitudinal stress over the cross-section $a\ b\ c\ d$ of a beam, where the maximum compression is found in the top fibres and maximum tension in the bottom, both uniformly diminishing to 0 in the plane of the neutral axis $N\ A$ of cross-section. Fig. 502 shows the corresponding distribution of the transverse shear over the same section, here being enclosed within the parabola $a\ f\ c$, with a maximum intensity at $N\ A$ and O shear at top and bottom of the section. Here $a\ e = \frac{2}{3} N\ f$ = average shear. Since the maximum intensity of shear occurs at the same place as zero longitudinal stress, it is safe to assume uniform distribution of the transverse shear.

Then $W = A\ S$ = maximum safe transverse shear in tons. .(46)

$$A = \frac{W}{S} = \text{minimum safe sectional area in square inches.} (47)$$

For a steel rod:

$$W = \frac{\pi d^2 S}{4} = 0.7854\, d^2 S = \text{maximum safe resistance in tons.} \quad (48)$$

$$A = \frac{W}{S} = \text{minimum safe sectional area of rod.} \quad . \quad . \quad . \quad (49)$$

$$d = 0.4605 \sqrt{W} = \text{minimum safe diameter of rod.} \quad . \quad . \quad (50)$$

438. Shear, Longitudinal.—Occurs only in wooden timbers parallel to the fibres. It is also assumed to be uniformly distributed.

Then $W = A\,S'$ = maximum safe shear in tons. . . . (51)

$A = \dfrac{W}{S'}$ = minimum safe area in square inches. . . . (52)

439. Compression in Short Struts (Fig. 503).—Short members in compression lengthwise, whose lengths do not exceed 5 times their least diameter or side of cross-section. Formulas are similar to those given for tension.

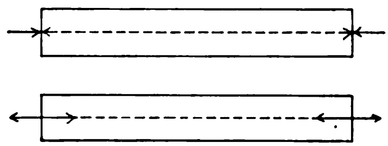


FIG. 503.

Then $W = A\,C$ = maximum safe compression in tons. . (53)

$A = \dfrac{W}{C}$ = minimum safe sectional area in square inches. (54)

For steel rods or cylinders:

$W = 0.787\,d^2\,C$ = maximum safe compression in tons. . (55)

$d = 1.128\sqrt{\dfrac{W}{C}}$ = minimum safe diameter in inches. . . (56)

Members with lengths exceeding 5 times their least diameter of side are usually subject to both compression and flexure, their safe strength being proportionately diminished.

440. Formulas for Columns and Posts.—The following straight line formulas are now commonly employed in practice, are safe within ordinary limits, and are far more convenient for use than the complex formulas of Euler, Hodgkinson, Rankine, Gordon, Ritter, etc.

For steel columns or posts:

$P = 8.00 - 0.42\,\dfrac{L}{R°}$ = maximum safe resistance in tons per sq. inch. . . . . (57)

P must not exceed 7.00 tons according to Chicago ordinance. (58)

$R°$ is the least radius of gyration of cross-section of column.

For steel columns filled and encased with concrete:

$P = 9.00 - 0.42\,\dfrac{L}{R°}$ = maximum safe resistance in tons per sq. inch. . . . . (59)

This is permitted by the Chicago ordinance.

For wrought iron columns and posts:

$$P = 6.00 - 0.36 \frac{L}{R^{\circ}} = \text{maximum safe resistance in tons per sq. inch.} \quad (60)$$

For cast iron columns, posts, or pilasters:

$$P = 5.00 - 0.24 \frac{L}{R^{\circ}} = \text{maximum safe resistance in tons per sq. inch.} \quad (61)$$

For wooden columns or posts:

$$P = C\left(1 - 0.15 \frac{L}{d}\right) = \text{maximum safe resistance in tons per sq. in.} \quad (62)$$

Take the value of C from the preceding Table of Coefficients.

L in feet must not exceed $2\frac{1}{2}$ times d in inches.

441. Uses of the Preceding Formulas.—These formulas likewise apply to straight principals and struts of roof trusses, subject only to longitudinal compression and not required to support transverse loads at the same time. Curved truss members in compression and those supporting purlins, etc., will be treated later under Compound Stresses.

442. Compression of Timbers across Fibres.—Formulas are similar to those already given.

$$W = A\ C' = \text{maximum safe compression in tons.} \quad . \quad . \quad . \quad (63)$$

$$A = \frac{W}{C'} = \text{minimum safe area in square inches.} \quad . \quad . \quad . \quad (64)$$

443. Transverse Loads on Beams, etc. (Figs. 504, 505, 506).—The member is either horizontal like a joist, beam, or purlin, or it may be inclined like a rafter or principal. In either case both ends are supported and it supports a loading arranged in various ways, either acting vertically or inclined to a vertical. It is simplest and generally advisable in practice to regard the member as cut at each end support, even if it does extend continuously over several spans.

444. Requirements for Safety.—A member supporting a transverse loading must always be safe from any danger of breaking near the middle of its span, and also from too great a deflection there, sufficient to crack plastering or to be unsightly. Hence two different and independent series of formulas are required, both of which must be applied in any given problem.

In the formulas for safety against breaking, the coefficient F in the table = maximum safe fibre stress in tons per square inch = compression or tension in the fibres most distant from the neutral axis of the cross-section.

445. Maximum Safe Deflection.—In the formulas for safety against bending too much, the maximum permissible deflection is taken at its ordinary value of $\frac{3}{100} L$ in inches.

The formulas here given are for three general cases, according to the arrangement of the loading on the member, supported at each end. They are derived from the general formulas usually given in the text-books, but are simplified in accordance with the notation at the beginning of this chapter. Both the formulas for breaking and bending must be applied to any problem, and the safest result is to be taken.

446. Case 1.—Load concentrated at middle of the span.

a. Breaking:

$$\frac{I}{c} = \frac{3\,W\,L}{F} = \text{minimum safe section modulus.} \quad (65)$$

$$W = \frac{F\,I}{3\,L\,c} = \text{maximum safe load in tons.} \quad (66)$$

$$L = \frac{F\,I}{3\,W\,c} = \text{maximum safe length in feet.} \quad (67)$$

b. Bending:

$$I = \frac{1080\,W\,L^2}{E} = \text{minimum moment of inertia of section.} \quad (68)$$

$$W = \frac{E\,I}{1080\,L^2} = \text{maximum safe load in tons.} \quad (69)$$

$$L = \sqrt{\frac{E\,I}{1080\,W}} = \text{maximum safe length in feet.} \quad (70)$$

447. Simplified Formulas for Steel.—These formulas may be further simplified for any particular material. For example, for rolled steel shapes, substituting 8.00 for F and 14500 for E and simplifying, the following formulas are obtained:

a. Breaking:

$$\frac{I}{c} = \frac{3\,W\,L}{8} = 0.375\,W\,L = \text{minimum safe section modulus.} \quad (71)$$

$$W = \frac{8\,I}{3\,L\,c} = 2.667\,\frac{I}{L\,c} = \text{maximum safe load in tons.} \quad . \quad . \quad (72)$$

$$L = \frac{8\,I}{3\,W\,c} = 2.667\,\frac{I}{W\,c} = \text{maximum safe length in feet.} \quad . \quad . \quad (73)$$

b. **Bending:**

$$I = 0.0745\,W\,L^2 = \text{minimum safe moment of inertia of section.} \; (74)$$

$$W = 13.426\,\frac{I}{L^2} = \text{maximum safe load in tons.} \quad . \quad . \quad . \quad . \quad (75)$$

$$L = 3.664\,\sqrt{\frac{I}{W}} = \text{maximum safe length in feet.} \quad . \quad . \quad . \quad (76)$$

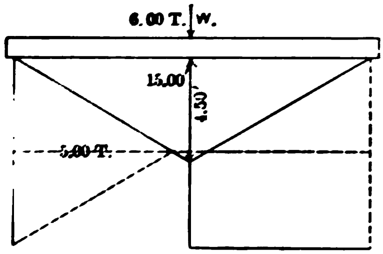


Fig. 504.

Values of section modulus and moment of inertia are given in Cambria and Carnegie, making it very easy to select the required shapes.

448. Example.—For example, Fig. 504 represents a member 15 ft. in clear span, loaded at the middle of span with 6 tons.

Hence $\frac{I}{c} = \frac{3 \times 6 \times 15}{8} = 33.75$, requiring 2, 9″ 21 # I-beams.

$I = 0.745 \times 6 \times 15^2 = 100.6$, requiring 2, 8″ 18 # I-beams.

Therefore, as the safest, 2, 9″ 21 # I-beams are to be used.

But a transverse shear of 3 tons occurs at each side of the middle of the beam, requiring $\frac{3.00}{6.00} = 0.50$ sq. in. to resist shear. Then $\frac{0.50}{9} = 0.0556$ in. = thickness of thin rectangular area of section for resisting shear, taken from section at middle of beam. For this rectangle, $.0556 \times 9.00$ in., $\frac{I}{c} = \frac{b\,d^2}{6} = \frac{.0556 \times 9^2}{6} = 0.68$. Subtracting this from the tabular value of $\frac{I}{c}$, $18.9 - 0.68 = 18.22$, which still exceeds the required value of 16.88 obtained by the formula. Hence the two I-beams are entirely safe against breaking and shear, acting together.

449. Case 2. Load Uniform over Entire Span.—

a. Breaking:

$$\frac{I}{c} = \frac{3\,W\,L}{2\,F} = \text{minimum safe section modulus.} \quad . \quad . \quad . \qquad (77)$$

$$W = \frac{2\,F\,I}{3\,L\,c} = \text{maximum safe load in tons.} \quad . \quad . \quad . \quad . \qquad (78)$$

$$L = \frac{2\,F\,I}{3\,Wc} = \text{maximum safe length in feet.} \quad . \quad . \quad . \qquad (79)$$

b. Bending:

$$I = \frac{675\,W\,L^2}{E} = \text{minimum safe moment of inertia of section.} \quad . \qquad (80)$$

$$W = \frac{E\,I}{675\,L^2} = \text{maximum safe load in tons.} \quad . \quad . \quad . \quad . \quad . \qquad (81)$$

$$L = .0385\sqrt{\frac{E\,I}{W}} = \text{maximum safe length in feet.} \quad . \quad . \quad . \qquad (82)$$

450. Simplified Formulas for Steel Beams.—These general formulas may also be simplified for steel as follows:

a. Breaking:

$$\frac{I}{C} = \frac{3\,W\,L}{16} = \text{minimum safe section modulus.} \quad . \quad . \quad . \quad . \qquad (83)$$

$$W = \frac{16\,I}{3\,L\,c} = \text{maximum safe load in tons.} \quad . \quad . \quad . \quad . \quad . \qquad (84)$$

$$L = \frac{16\,I}{3\,W\,c} = \text{maximum safe length in feet.} \quad . \quad . \quad . \quad . \quad . \qquad (85)$$

b. Bending:

$$I = .0466\,W\,L^2 = \text{minimum safe moment of inertia of section.} \qquad (86)$$

$$W = 21.481\,\frac{I}{L^2} = \text{maximum safe load in tons.} \quad . \quad . \quad . \quad . \quad . \qquad (87)$$

$$L = 4.635\sqrt{\frac{I}{W}} = \text{maximum safe length in feet.} \quad . \quad . \quad . \quad . \qquad (88)$$

451. Example.—Fig. 505 exhibits a uniform load of 6 tons on a beam 15 ft. span. The moment and shear diagrams are given in the figure. Applying the formulas just given for $\frac{I}{c}$ and for I:

$\frac{I}{c} = \frac{3 \times 6 \times 15}{16} = 16.9$. Hence 2, 7″ 15 # I-beams are required.

$I = 0.0466 \times 6 \times 15^2 = 62.9$. Hence 2, 7″ 15 # I-beams are required.

These beams will suffice for both breaking and bending, and the shear may be neglected, since no shear occurs at the same point as the maximum bending moment.

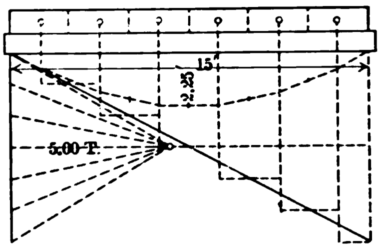


FIG. 505.

452. Simplified Formulas for Joists, Rafters, etc.—If the preceding general formulas for Case 2 are to be applied to joists or rafters, they may be changed to a more convenient general form.

Let e = distance between centres of joists or rafters in inches.

w = sum of live and dead loads in lbs. per square foot of floor.

a. Breaking:

$$\frac{I}{c} = \frac{L^2\,e\,w}{16000\,F} = \text{minimum safe section modulus.} \qquad (89)$$

$$w = \frac{16000\,F\,I}{L^2\,e\,c} = \text{maximum safe load in lbs. per sq. ft.} \qquad (90)$$

$$e = \frac{16000\,F\,I}{L^2\,w\,c} = \text{maximum safe distance on centres in inches.} \qquad (91)$$

$$L = 126.5\sqrt{\frac{F\,I}{w\,e\,c}} = \text{maximum safe length in feet.} \qquad (92)$$

b. Bending:

$$I = \frac{w\,L^3\,e}{35.56\,E} = \text{minimum safe moment of inertia of section.} \qquad (93)$$

$$w = \frac{35.56\,E\,I}{L^3\,e} = \text{maximum safe load in lbs. per square foot.} \qquad (94)$$

$$e = \frac{35.56\,E\,I}{L^3\,w} = \text{maximum safe distance on centres in inches.} \qquad (95)$$

$$L = 3.29\sqrt[3]{\frac{E\,I}{w\,e}} = \text{maximum safe length in feet.} \qquad (96)$$

453. Simplified Formulas for Steel Joists or Rafters.—The last general formulas may also be simplified for steel joists or rafters in the following form:

a. Breaking:

$$\frac{I}{c} = \frac{w\,L^2\,e}{128000} = \text{minimum section modulus.} \quad (97)$$

$$w = \frac{128000\,I}{L^2\,e\,c} = \text{maximum safe load in lbs. per square foot.} \quad (98)$$

$$e = \frac{128000\,I}{L^2\,w\,e} = \text{maximum safe distance on centres in inches.} \quad (99)$$

$$L = 357.8\sqrt{\frac{I}{w\,e\,c}} = \text{maximum safe length in feet.} \quad (100)$$

b. Bending:

$$I = \frac{w\,L^3\,e}{515556} = \text{minimum safe moment of inertia of section.} \quad (101)$$

$$w = \frac{515556\,I}{L^3\,e} = \text{maximum safe load in lbs. per sq. ft.} \quad (102)$$

$$e = \frac{515556\,I}{L^3\,w} = \text{maximum safe distance on centres in inches.} \quad (103)$$

$$L = 80.185\sqrt[3]{\frac{I}{w\,e}} = \text{maximum safe length in feet.} \quad (104)$$

454. Simplified Formulas for Wooden Flooring or Sheathing.—The general formulas for this case may also be simplified for wooden flooring or sheathing, to determine its thickness, maximum safe load, or distance in feet between centres of supporting beams, rafters, or joists.

a. Breaking:

$$t = \frac{L}{51.6}\sqrt{\frac{w}{F}} = \text{least safe thickness in inches.} \quad (105)$$

$$w = 2667\,\frac{F\,t^2}{L^2} = \text{maximum safe load in lbs. per sq. ft.} \quad (106)$$

$$L = 51.6\,t\sqrt{\frac{F}{w}} = \text{maximum safe length in feet.} \quad (107)$$

b. Bending:

$$t = \frac{L}{1.44}\sqrt{\frac{w}{E}} = \text{minimum safe thickness in inches.} \quad (108)$$

$$w = 2.96\,\frac{E\,t^3}{L^3} = \text{maximum safe load in lbs. per sq. ft.} \quad (109)$$

$$L = 1.44\, t \sqrt[3]{\frac{E}{w}} = \text{maximum safe length in feet.} \quad . \quad . \quad (110)$$

455. Case 3. Loading Arranged in any Manner.—Fig. 506 shows such a beam loaded in an irregular manner.

The maximum value of the bending moment M in foot-tons occurring at any point in the length of the beam is to be first found, either graphically, as in the figure, or by computation.

a. Breaking:

$$\text{Then } \frac{I}{c} = \frac{12\, M}{F} = \text{minimum section modulus.} \quad . \quad . \quad . \quad . \quad (111)$$

$$M = \frac{F\, I}{12\, c} = \text{maximum safe bending moment in foot-tons.} \quad (112)$$

b. Bending:

No formulas exist that are directly applicable to any case of irregular loading. But two series of formulas give the limiting values corresponding to a load concentrated at the middle, as in Case 1, and to a uniform load, as in Case 2. The actual values are then to be taken within these limiting values, according as the arrangement of the loading approximates to a concentrated or a uniform load.

456. Limiting Formulas for Deflection.—1. Loading concentrated at middle of span:

$$I = \frac{4320\, M\, L}{E} = \text{minimum moment of inertia of section.} \quad . \quad (113)$$

$$M = \frac{E\, I}{4320\, L} = \text{maximum safe bending moment in ft.-tons.} \quad (114)$$

$$L = \frac{E\, I}{4320\, M} = \text{maximum safe length in feet.} \quad . \quad . \quad . \quad . \quad (115)$$

2. Loading uniformly distributed over span:

$$I = \frac{5400\, M\, L}{E} = \text{minimum safe moment of inertia of section.} \quad (116)$$

$$M = \frac{E\, I}{5400\, L} = \text{maximum safe bending moment in ft.-tons.} \quad . \quad (117)$$

$$L = \frac{E\, I}{5400\, M} = \text{maximum safe length in feet.} \quad . \quad . \quad . \quad . \quad . \quad . \quad (118)$$

In any practical case it is well to use the safest value obtained by use of these limit formulas for bending.

457. Simplified Formulas for Steel Beams.—The formulas for Case 3 may likewise be simplified for application to rolled steel shapes, as before suggested.

a. Breaking:

$$\frac{I}{c} = 1.5\,M = \text{minimum safe section modulus.} \quad . \quad . \quad . \quad . \qquad (119)$$

$$M = \frac{I}{1.5\,c}\ \text{maximum safe bending moment in ft.-tons.} \quad . \quad . \qquad (120)$$

b. Bending.

1. Load concentrated at middle of span:

$$I = .2979\,M\,L = \text{minimum safe moment of inertia of section.} \qquad (121)$$

$$M = 3.356\,\frac{I}{L} = \text{maximum safe bending moment in foot-tons.} \qquad (122)$$

$$L = 3.356\,\frac{I}{M} = \text{maximum safe length in feet.} \quad . \quad . \quad . \quad . \qquad (123)$$

2. Loading uniformly distributed over span:

$$I = .3724\,M\,L = \text{minimum safe moment of inertia of section.} \qquad (124)$$

$$M = 2.685\,\frac{I}{L} = \text{maximum safe bending moment in ft.-tons.} \qquad (125)$$

$$L = 2.685\,\frac{I}{M} = \text{maximum safe length in feet.} \quad . \quad . \quad . \quad . \qquad (126)$$

458. Example.—Fig. 506 exhibits an irregularly loaded beam supporting concentrated loads of 2 and 3 tons with a partial uniform load of 3 tons. Here $M = 13.25$ foot-tons and is found just above the intersection of shear line and shear axis.

a. Breaking:

$$\frac{I}{c} = \frac{12\,M}{F} = \frac{12 \times 13.25}{8} = 19.88.$$

b. Bending.

1. Loading concentrated:

$$I = \frac{4320\,M\,L}{E} = \frac{4320 \times 13.25 \times 15}{14500} = 59.2.$$

2. Loading uniform:

$$I = \frac{5400\,M\,L}{E} = \frac{5400 \times 13.25 \times 15}{14500} = 74.0.$$

For breaking: 2, 7″ 15 # I-beams are sufficient.

For bending: 2, 7″ 15 # I-beams suffice for concentrated load.

For bending: 2, 7″ 17½ # I-beams suffice for uniform load.

Hence it will be safest to use 2, 17½ # I-beams.

In applying the formulas of Cases 1, 2, and 3, labor and time will be saved and errors avoided by using a good table of four-place logarithms. Huntington's Four Place Tables are very convenient and sufficiently accurate.

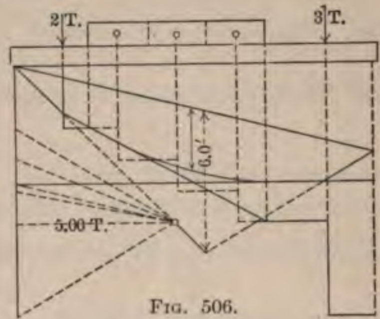


FIG. 506.

459. Compound Stresses.—Stresses of different kinds frequently act on a structural member at the same time. A single formula may sometimes be deduced to provide for all these stresses in dimensioning the member, but generally a separate formula is to be applied for each stress, combining their results in determining the final dimensions of the section of the member.

460. Transverse Shear and Bending Moment.—This always occurs in a loaded beam, and the section required by the bending moment is first determined as in Case 3; the sectional area for resisting shear is then found; the first section then being increased by the addition of the latter. Shear may generally be neglected in beams with uniform loading.

461. Tension and Bending Moment.—

462. Axis of Member Straight (Fig. 507.)—The longitudinal tension is assumed to have uniform intensity over the cross-section of the member, which is also required to support three loads, producing a bending moment. This is the case of the lower chord of a roof truss, which also supports ceiling beams or joists.

Let X = total tension in tons acting in the member.

M = maximum bending moment in foot-tons acting on it.

A = sectional area in square inches of the member.

c' = distance in inches from neutral axis to top fibre of section.

c'' = distance in inches from neutral axis to bottom fibre.

For sections symmetrical about the neutral axis, $c' = c'' = \frac{\text{depth}}{2}$.

I = moment of inertia of cross-section of member.

Then $\frac{X}{A} - \frac{Mc'}{I}$ = maximum stress in top fibres of section, tons

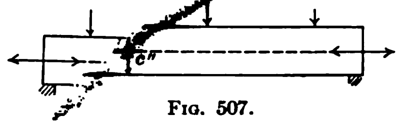


FIG. 507.

per sq. in. . . . . (127)

$\frac{X}{A} + \frac{Mc''}{I}$ = maximum stress in bottom fibres, tons per sq. in. (128)

This maximum stress must not exceed C for upper or T for lower fibres, as given in table of coefficients for materials.

It is evidently necessary to first assume the section of the member, then applying the formula, proceeding in this manner until the safe section be found.

463. Axis of Member Curved (Fig. 508).—Member in tension only. Axis is assumed to be a circular arc, convex upward, like the curved chord of a truss.

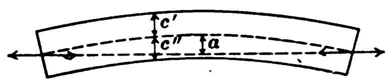


FIG. 508.

Let a = versed sine of longitudinal curved axis in feet.

Then $M = Xa$ = bending moment in foot-tons due to curvature of axis.

$\frac{X}{A} - \frac{Xac'}{I}$ = maximum stress in tons per sq. in. in top fibres. (129)

$\frac{X}{A} + \frac{Xac''}{I}$ = maximum stress in tons per sq. in. in bottom fibres. (130)

These maximum stresses must not exceed C or T in the table.

464. Axis of Member Curved with Bending Moment (Fig. 509).—

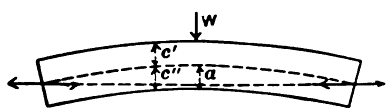


FIG. 509.

Then $\frac{X}{A} - \frac{Xac'}{I} + \frac{Mc'}{I}$ = maximum stress in top fibres. . (131)

$$\frac{X}{A}+\frac{X a c''}{I}-\frac{M c''}{I} = \text{maximum stress in bottom fibres.}$$

These must not exceed the safe values of C and T in table.

465. Compression and Bending Moment.—

466. Axis of Member Straight (Fig. 510).—Let Z = total compression in tons acting in the member. Intensity uniform entire cross-section.

Then $\frac{Z}{A}+\frac{M c'}{I}$ = maximum stress in top fibres. . . . . (

$$\frac{Z}{A}-\frac{M c''}{I} = \text{maximum stress in bottom fibres.} \quad . \quad . \quad (1$$

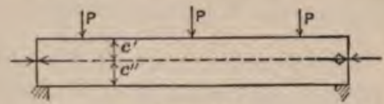


FIG. 510.

467. Axis of Member Curved (Fig. 511).—

$$\frac{Z}{A}+\frac{Z a c'}{I} = \text{maximum stress in top fibres} \quad . \quad . \quad . \quad (13$$

$$\frac{Z}{A}-\frac{Z a c''}{I} = \text{maximum stress in bottom fibres.} \quad . \quad . \quad (13$$

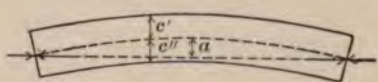


FIG. 511.

468. Axis of Member Convex Upward; Bending Mome (Fig. 512).—

$$\frac{Z}{A}+\frac{Z a c'}{I}-\frac{M c'}{I} = \text{maximum intensity of stress in top fibres.} \quad (13$$

$$\frac{Z}{A}-\frac{Z a c''}{I}+\frac{M c''}{I} = \text{maximum intensity of stress in bottom fibres.} (13$$

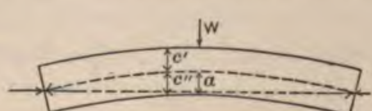


FIG. 512.

Values obtained by these formulas must not exceed the sa values of C and T respectively given in the table of coefficients.

469. Inclined Rafter (Fig. 513).—A rafter is supported at ea end, is assumed to extend over but one span, and usually suppo

a uniform load, which rarely acts at right angles to the axis of the rafter.

Let L = its inclined free length in feet.

i° = its inclination from a horizontal.

W = resultant in tons of total uniform load on it.

$L \cos i^\circ$ = horizontal projection of rafter in feet.

W' = component of W acting at right angles to rafter.

W'' = component of W acting parallel to rafter.

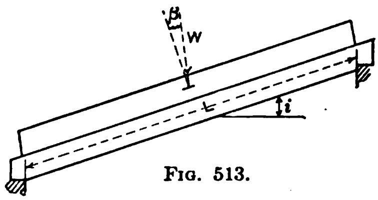


FIG. 513.

The component W' produces bending and deflection; the component W'' causes longitudinal compression in the rafter, and it has the value $\frac{W''}{2}$ at the mid-length of the rafter, where the maximum deflection occurs. While the rafter remains straight, this compression is uniformly distributed over its cross-section. Hence $\frac{W''}{2A}$ = intensity of this compression in tons per square inch. . . (139)

But the component W' produces at the same time a deflection at middle of rafter.

470. Maximum Fibre Stress.—

$\Delta = \frac{22.5\, W' L^3}{E I}$ = maximum deflection in ins. at middle. (140)

Then $\frac{W''}{2A}\left(1 + \frac{6\,\Delta}{d}\right)$ = maximum tensile fibre stress in tons per sq. in. at bottom of section of rafter at middle. . . . . (141)

Here d = depth in ins. of rafter, for rectangular, I or channel section, or for any other form symmetrical about its neutral axis.

Then F − this maximum fibre stress = safe value for F to be substituted in formula for $\frac{I}{c}$ in Case 2, for computing anew the value of $\frac{I}{c}$.

It is usually found in practice that this parallel component W'' can be safely neglected, excepting in case of very steep roofs.

For rapid approximate calculations, the total load W is often assumed to be uniformly distributed over a horizontal beam of the length $L \cos i°$, or the horizontal projection of the rafter.

471. Horizontal Beam Resisting Maximum Bending Moment and Cross Shear (Figs. 504, 505, 506).—The cross-section of the beam is first found for safely resisting the maximum bending moment M by the formulas of Case 3, according to the arrangement of the loading. The shear diagram then shows the intensity of transverse shear occurring at or near the same point as the maximum moment M. The formula for shear then gives the area of section to be added to the cross-section already found, for resisting this shear, if necessary.

Transverse shear can usually be neglected for uniform loading, if the sections of the beam are uniform, being $= 0$ at middle of beam.

Longitudinal shear has a maximum value at the middle of the depth of wooden beams, but it may usually be neglected, excepting for heavily loaded short and deep beams, which are rarely found in roof construction.

472. Purlins Supporting Rafters or Sheathing. — These are usually horizontal and straight, set with sides or main axis of section vertical, perpendicular to inclination of roof, or radial for curved roofs.

Let W = total resultant in tons of uniform load on purlin, which may coincide with main axis of its cross-section, or may make with this an angle α. (Fig. 515.)

Two cases comprise all positions of the load and purlin.

473. Resultant of Loads Coincides with Main Axis.—W coincides with main axis of its section. Maximum fibre stress will not exceed safe value of F. (Fig. 514.)

474. Resultant of Loads Does Not so Coincide.—W makes an angle α with the main axis. Maximum fibre stress may exceed safe value of F, and further calculations are required.

$X X$ is here the main axis of the cross-section.

$Y Y$ is its transverse axis.

Then $W' = W \cos \alpha$ = component coincident with main axis $X X$. . . . . (142)

$W'' = W \sin \alpha$ = component coincident with transverse axis $Y Y$. . . . . (143)

By formulas for Case 2, select a cross-section possessing the required numerical values of $\frac{I}{c}$ and I about the axes $Y\ Y$ and $X\ X$, respectively, as for example in Fig. 515.

475. Location of Neutral Axis of Section.—The gravity axis at which 0 stress occurs may be located by the following method. In Fig. 515, at any convenient scale, lay off $C\ D$ = numerical value of I_y about axis $Y\ Y$ and $C\ B$ = numerical value of I_x about axis $X\ X$. Draw horizontal $D\ A$ and through B draw $B\ A$ perpendicular to $C\ W$ and intersecting at A the horizontal through D. Join $A\ C$, which is the required line of 0 stress.

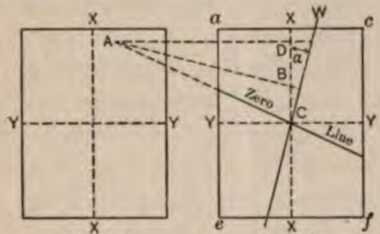


Fig. 514   Fig. 515

It is here evident that the compression at c and the tension at e will be equal and greater than in any other fibres of the section. Their intensity may be found by the following formulas.

Let d = depth of purlin in ins. parallel to main axis $X\ X$.

Let b = breadth of purlin in ins. parallel to axis $Y\ Y$.

$W' = W \cos a$ = component of W parallel to axis $X\ X$.

$W'' = W \sin a$ = component of W parallel to axis $Y\ Y$.

476. Formulas for Maximum Fibre Stress.—Case 1. Load W concentrated at mid-length of purlin.

$$1.5\, L \left(\frac{W' d}{I_y} + \frac{W'' b}{I_x}\right) = \text{maximum fibre stress in tons per sq. in.} \quad (144)$$

Case 2. Load W uniformly distributed along purlin.

$$.75\, L \left(\frac{W' d}{I_y} + \frac{W'' b}{I_x}\right) = \text{max. fibre stress in tons per sq. in.} \quad (145)$$

Case 3. Load W arranged on purlin in any manner.

$$6\left(\frac{M' d}{I_y} + \frac{M'' b}{I_x}\right) = \text{max. fibre stress in tons per sq. in.} \quad (146)$$

Here $M' = M \cos a$ = moment of component parallel to axis $X\ X$.

$M'' = M \sin a$ = moment of component parallel to axis $Y\ Y$.

477. Use of these Formulas.—These formulas are applicable to any section symmetrical about the gravity axis $Y\,Y$, such as timbers, I-beams, channels, box girders, etc., but not to angles and tees.

The value obtained for fibre stress by either formula will frequently exceed the safe value for F, when a larger section must be chosen, then again applying the formula to this, until one is found whose maximum fibre stress does not exceed F for the material in the table of coefficients.

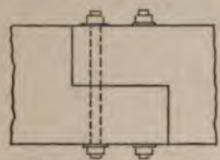


Fig. 516.

478. Spliced Timbers in Compression (Fig. 516).—The compressile stress in the member (principal or strut) is assumed to be uniform and transmitted by direct contact. Hence a simple and short halved splice is sufficient, the timbers being held together by two to four bolts. To prevent the bending of the member, this splice should be made as near an apex as possible.

479. Spliced Timbers in Tension (Fig. 517).—These splices are to be made as simple as possible, to economize labor and materials. Therefore butt splices with vertical steel fish-plates and through bolts are preferred and employed here.

480. Sectional Dimensions of Spliced Timber in Tension.—Experiments made by Mr. John C. Gustafson proved that the ultimate resistance of such splices, made in various ways, was from 50 to 70 per cent of that of the uncut timbers. Hence the safe strength of a tension splice is here assumed at 50 per cent of that for the timber, and in dimensioning the member in tension, the area of section found by using the coefficient T is doubled.

Let X = tensile stress in tons acting in the member to be spliced.
b = horizontal breadth in inches of the cross-section.
h = vertical depth in inches of the section.

$$\text{Then } A = \frac{2\,X}{T} = \text{minimum area of its section in sq. ins.} \quad . \quad (147)$$

481. Number of Rows of Bolts in Splice.—

Let n = number of horizontal rows of bolts in the splice.
$n = 1$ for timbers 4 to 6 inches deep.
$n = 2$ for timbers 8 to 12 inches deep.
$n = 3$ for timbers 14 to 18 inches deep.
$n = 4$ for timbers 20 to 24 inches deep.

To locate horizontal centre lines of the bolts:

For $n = 1$, divide h into 2 equal parts; centres are on middle line.

For $n = 2$, divide h into 4 equal parts; centres on 1st and 3d lines.

For $n = 3$, divide h into 6 equal parts; centres on 1st, 3d, and 5th lines.

For $n = 4$, divide h into 8 equal parts; centres on 1st, 3d, 5th, and 7th.

This arrangement of the bolts uniformly distributes the tensile stress over the entire uncut cross-section of the timber.

482. Formulas for Splices of Timbers in Tension (Fig. 517).—
Let N = number of bolts required in one end of splice.

d = diameter in inches of the bolts used.

t = thickness in inches of the fish-plate.

g = minimum horizontal distance between centres of bolts or from centre of bolt to end of timber.

C = coefficient of safe resistance of wood to crushing endwise.

S' = coefficient of safe resistance to shear parallel to fibres.

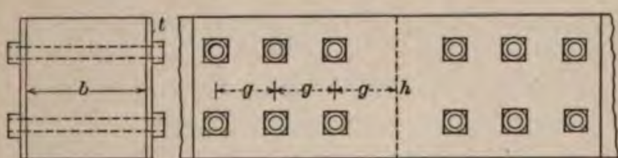


FIG. 517.

The uncut total cross-section of the fish-plates must transfer the stress in the member; the thickness of each plate must be sufficient to develop the full resistance to shear in end of each bolt; the resistance to shear at each end of the bolt, the resistance of the wood to crushing by the bolt, and also the resistance to shearing endwise of the wood at each side of the bolt, are arranged to equal each other in the formulas given here.

$g = \dfrac{dC}{2S'}$ = minimum distance of bolts on centres or to end of timber. (148)

$t = \dfrac{3}{8}d$ = minimum safe thickness of fish-plate to develop full resistance of bolt to shearing. . . . . (149)

$t' = \dfrac{X}{16\,(h - nd)}$ = minimum thickness of fish-plate for transmitting the tensile stress X in the member. . . . . (150)

The two formulas for t being independent of each other, the larger value obtained by applying them is to be used.

Holes in fish-plates for bolts must always be drilled, never punched.

$N = \dfrac{X}{b\,d\,C}$ = number of bolts required in each end of splice. (151)

This formula is based on the condition that the thickness of the plate is obtained by formula 149. But if t' be less than t, and this value be used for any reason, the following formula should be applied, which increases the value of N to be used:

$N = \dfrac{t}{t'} \times \dfrac{X}{b\,d\,C}$ = number of bolts to be used in each end of splice. . . . . (152)

Diameter of bolts should be from $\frac{1}{2}''$ to $1''$, using the manufacturers' sizes as given in Cambria and Carnegie.

483. Formulas for Fish-Straps.—Separate fish-straps instead of fish-plates might also be used as in Fig. 518, with the advantage of permitting the shrinkage of the timbers without causing cracks.

Let h' = width in inches of one strap.

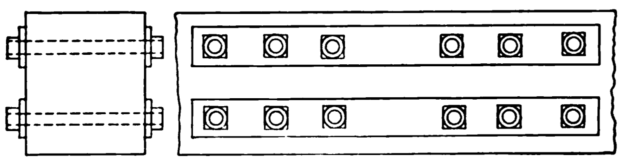


FIG. 518.

n = number of straps or rows of bolts.

$t = \dfrac{X}{16\,n\,(h' - d)}$ = minimum thickness of one strap, which should not be made less than $\frac{3}{8}\,d$. . . . . (153)

Or if $t = \frac{3}{8}\,d$, then $h' = \dfrac{X + 6\,n\,d^2}{6\,n\,d}$ . . . . (154)

484. Riveted Connections.—Dimensions of rivets and rivet heads are given in Cambria, page 342, and Carnegie, page 191.

Safe shearing and bearing resistances of rivets are to be found in Cambria, pages 316, 317, and in Carnegie, pages 195, 196. The new Chicago building ordinance permits for shop rivets 6 tons per square inch of section in single shear and 12.5 tons per square inch in bearing. For field rivets, 5 tons in single shear and 10 tons in bearing. These values are entirely safe in roof trusses, etc. See table on page 195 of Carnegie. (Not in Cambria.)

Holes for rivets are punched by a machine making a group of holes at the same time, making it necessary to arrange the holes in accordance with the standard punching used in the shops. Standard punching for connections are in Cambria, pages 51–54, and in Carnegie, pages 177, 178.

485. Spacing of Rivets.—Rules for spacing rivets are to be found in Cambria, page 322. The rules and data given in the two handbooks vary slightly, according to the system employed in the shop executing the work.

Rivets $\frac{3}{4}$ inch in diameter are generally used throughout ordinary roof trusses, or $\frac{7}{8}$ inch for very heavy work. Different sizes in a truss should be avoided on account of lost time in changing punches, dies, and rivets.

For accurately obtaining the total distance between end centres of a string of rivets, Smoley's table, pages 316–318, will be found useful.

486. Location of Rivet Lines.—Theoretically, the longitudinal gravity axis of a member composed of two rolled shapes should coincide with the centre line of the member on the truss diagram. But it is practically more convenient and usual to so arrange the member that either a rivet centre line coincides with the centre line of a member on the truss diagram, or that this centre line of the member lies midway between two rivet centre lines. Resulting bending moments at the apexes are then small and may usually be neglected.

487. Clearance for Rivet Heads.—For all angles must be allowed $\frac{1}{4}$ or $\frac{3}{8}$ inch for clearance between rivet head and leg of angle, if rivets are staggered or are set in one leg only. Add to this the height of head, in chain riveting in both angles.

Minimum distance from back of angle to rivet centre line.

a. Chain riveting in both legs:

Let t = thickness of leg.

d' = diameter of head of rivet.

h = height of head.

Then $t + h + \frac{d'}{2} + \frac{1}{4}''$ or $\frac{3}{8}''$ = this minimum distance. . (155)

($\frac{1}{4}$ inch is given in Carnegie and $\frac{3}{8}$ inch in Cambria, which is more common.)

b. Staggered riveting in legs:

$t + \frac{d'}{2} + \frac{1}{4}''$ or $\frac{3}{8}''$ = this minimum distance. . . . (156)

Distance to rivet centre line is always figured from back of angle and not from the leg.

488. Standard Arrangement of Rivet Lines Arranged.—But rivet centre lines on angles are generally spaced according to the table of the American Bridge Co., without regard to thickness of the leg.

TABLE FOR SPACING RIVET CENTRE LINES ON ANGLES (FIG. 519).

Leg	G.	G_1.	G_2.	G_3.	Max. rivet.
2″	1⅛″	...	...	...	½″
2¼″	1¼″	...	...	...	⅝″
2½″	1⅜″	...	...	...	⅝″
2¾″	1⅝″	...	...	...	¾″
3″	1¾″	...	...	...	⅞″
3½″	2″	...	...	...	⅞″
4″	2¼″	...	...	...	⅞″
4½″	2½″	...	...	...	⅞″
5″	3″	2″	1¾″	1¼″	⅞″
6″	3½″	2¼″	2½″	1¼″	⅞″
7″	4″	2½″	3″	1½″	⅞″
8″	4½″	3″	3″	2″	⅞″

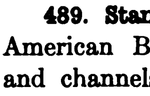


FIG. 519.

Two rows of rivets in a leg less than 5 inches wide must be staggered, and the rivets in the different legs should also be staggered, when possible. Chain riveting is preferable at apexes of trusses.

Standard punchings for connecting angles for I-beams, channels, etc., are given on pages 51–58 of Cambria, and pages 176–178 of Carnegie.

489. Standard Punching for I-Beams and Channels.—The American Bridge Co.'s standard punching in webs of I-beams and channels is given in Fig. 520 and should be used, since multiple punching machines are usually arranged to suit these spacings.

Pitch of rivets on centre line must not be less than 3 diameters and should never exceed 6 inches or 16 times the thickness of the thinnest plate connected by the rivets.

The safe resistance of a rivet is determined by its safe shear or bearing resistance, since its bending stress is usually smaller and is neglected.

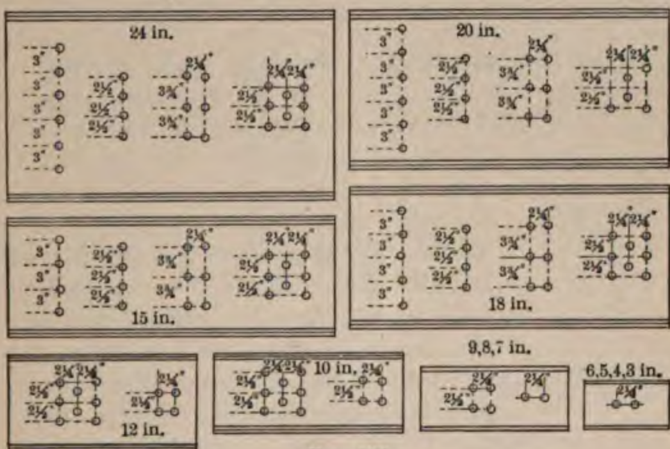


Fig. 520.

490. Spliced Channels in Compression.—Steel shapes are sometimes milled to exact lengths and with squared ends, but riveting is likely to separate these ends, so that all compression is assumed to be transmitted between the shapes through the rivets and cover plates or gussets. No deduction is made for rivet holes. (Fig. 521.)

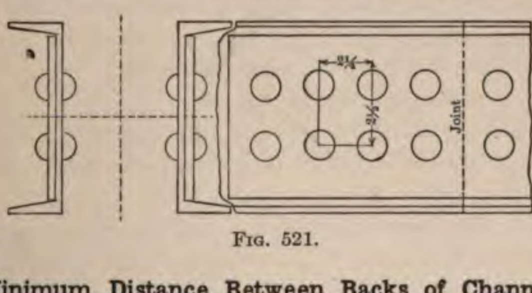


Fig. 521.

491. Minimum Distance Between Backs of Channels.—For a compression member, it first becomes necessary to determine the minimum clear distance between the backs of channels forming the member, so that the member may be equally stiff about both rect-

angular axes of the cross-section. This distance may be computed by the following formula:

$$2\left(\sqrt{R_1^2 - R_2^2} - x\right) = \text{minimum clear distance between backs.} \quad (157)$$

Here R_1 = radius of gyration about axis 1 1 (Fig. 522).

R_2 = radius of gyration about axis 2 2.

x = distance from centre of gravity to back of channel.

For these values see Cambria or Carnegie, Properties of sections.

492. Graphical Method for Obtaining this Distance.—This distance may also be found by a simple graphical construction (Fig. 522). For sake of clearness, Fig. 522 is drawn to twice the scale of Figs. 521 and 523. In Fig. 522, lay off $C\,b = R_2$ on horizontal drawn through C; also $C\,a = R_2$ on the same; erect the perpendicular $a\,c$ and with radius $C\,b$ describe arc $b\,c$; lay off $c\,d = x$, and $a\,d$ = one-half the required clear distance between the backs of the channels. But since Fig. 522 is at double the scale of the section in Fig. 521, $a\,d$ = clear distance required. These clear distances are to be found in Tables I and J, Figs. 540 to 552, written on line representing each channel.

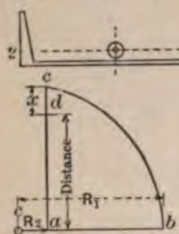


FIG. 522.

According to Fig. 520, two rows of rivets are to be used here, set $2\frac{1}{2}$ inches between centre lines, and pitch of rivets is $2\frac{1}{4}$ inches, with $1\frac{1}{2}$ inches from centre of rivet to end of cover plate or shape.

At a splice, the compressile stress is transmitted through flange and web cover plates, between which it is divided in proportion to the number of lines of rivets in flanges and webs.

Channels with gussets and not latticed.

The gusset here transmits one-half the stress, which would be transmitted by web cover plates of latticed channels. Number of rivets is found by Table T.

493. Spliced Channels in Tension.—Latticed channels are spaced the same distance apart as for channels in compression in Fig. 521, but they would have equal tensile strength if a gusset were placed between them with cover plates on each side of the web, and this would materially reduce the number of rivets required at a splice, since each rivet would be in quadruple shear.

Rivet holes in any cross-section must be deducted, taking the rivet $\frac{1}{8}$ inch larger to allow for reduction by punching the holes. Hence the cover plates will be a little thicker than for a splice transmitting an equal compressile stress. The number of rivets is determined by the bearing of the rivets on the thin web of the channel in Fig. 523, as this does not develop the full resistance of the rivet to single shear. Number of rivets required is found by Table T.

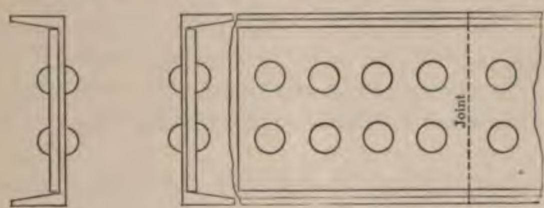


Fig. 523.

In heavy structures, cover plates are usually riveted on the flanges as well as on the webs at the splice.

The stress at the splice is divided between flange and web covers in proportion to the number of rivet lines in each.

494. Pin Connections.—Joint pins are cylindrical and must be finished straight and smooth with uniform diameter. To retain the connected members on the pin, its ends are turned smaller and receive standard hexagonal nuts, as in Cambria, pages 344, 345, Carnegie, page 200; or split steel cotters are sometimes inserted in holes at the ends of the pin. The finished diameter of the pin is $\frac{1}{16}$ inch less than the rough or nominal diameter, to allow for turning or cold rolling. Cold-rolled steel shafting is often used for pins and does not require finishing.

A pilot nut is placed on the end of the pin while inserting it in place and assembling the members, to prevent injury to screw threads.

495. Dimensions of Eye-bar Ends.—Standard dimensions of flat eye ends to connect on pins are given in Cambria, page 339, and in Carnegie, page 212. Since the plain flat bar is not adjustable in length, flat eyes with screw ends are also employed, being connected by a round rod with enlarged ends and turnbuckles or sleeve nuts. For standard dimensions, see Cambria, pages 338–342, also Carnegie,

pages 210–212. End of round rod must be upset as per Cambria, pages 334, 335, or Carnegie, pages 205, 206. For members to resist heavy stresses, plain flat eye-bars are generally used, carefully made to the exact lengths required. The adjustment of length is then made in other members.

Pins may fail in either of three ways, each of which usually requires to be considered.

496. Resistance to Shearing.—The maximum intensity of shear in a round section is at the middle diameter of the pin, where this exceeds the average intensity by one-third. But it is always assumed to be uniformly distributed over the section.

Let d = diameter of pin in inches.

Z = maximum longitudinal stress acting in any member connected, in tons.

Then $d = 0.4607\sqrt{Z}$ = diameter of pin for single shear. (160)

$d = 0.3257\sqrt{Z}$ = diameter of pin for double shear. (161)

Single shear occurs for the outer members acting on pins, and double shear on the others; hence members with smaller stresses should be placed nearest the ends of the pin. The middle member acting on the pin may be a single rod or shape, but all others must be doubled or have forked ends in order to produce symmetrical shears on the pin. It is usually best to place the web member in compression at the middle of the pin.

497. Resistance to Bearing or Crushing.—If the pin accurately fits the holes in the members, it is probable that their pressures on the pin act radially. Yet this pressure is always assumed to be uniformly distributed over the area $d\,t$, d being the diameter of the pin and t the length of the bearing surface of the member.

Then 12.5 tons per sq. in. = maximum safe resistance for bearing.

$12.5\,d\,t$ = maximum safe bearing resistance in tons. . (162)

$d = \dfrac{Z}{12.5\,t} = 0.8\dfrac{Z}{t}$ = diameter of pin with one bearing at end of member. . . . . (163)

$d = \dfrac{Z}{25.0\,t} = 0.4\dfrac{Z}{t}$ = diameter of pin with two bearings at end of member. . . . . (164)

$t = 0.377\,d$, approximately $\dfrac{3}{8}d$. . . . . . (165)

For flat eye-bars, t should equal $\dfrac{1}{6}$ to $\dfrac{1}{4}$ their width. . (166)

Table in Cambria, pages 398–403, or in Carnegie, pages 245–250, will be found very convenient in dimensioning plain eye-bars.

Also table for round rods with upset ends and eye ends, Cambria, pages 334, 335, and Carnegie, pages 261–266.

For square bars with upset ends, see Cambria, pages 336, 337, or Carnegie, pages 261–266.

$\frac{\text{Stress}}{8}$ = minimum sectional area of eye-bars composing the member.

Square bars are mostly used for diagonals of bridge trusses, rarely for roof trusses.

498. Resistance to Breaking.—Since the stresses are concentrated at different points in the length of the pin, employ formulas of Case 3.

Let M' = maximum bending moment in inch-tons acting on the pin.

$$d = 1.0839\sqrt[3]{M'} = 1\frac{1}{12}\sqrt[3]{M'} = \text{minimum safe diameter of pin.} \quad (167)$$

499. Comparison of Pin and Rivet Connections.—As an example, take the connections at the ends of member 7 8 in Example 4 of Chapter IV, Fig. 72, a steel Fink truss of 128 ft. span.

500. Pin Connections.—For pin connections it is most con-

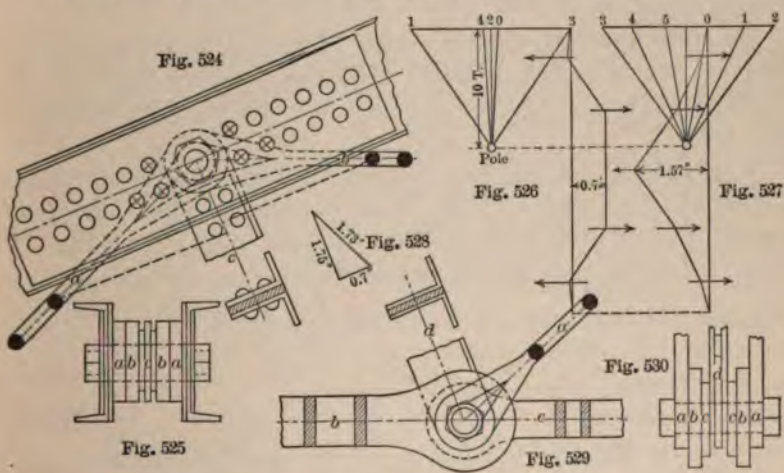


Fig. 524 Fig. 525 Fig. 526 Fig. 527 Fig. 528 Fig. 529 Fig. 530

venient to use two channels for the upper chord, since a pin connection in two angles would be awkwardly made. These channels are

set apart sufficiently to make the member equally stiff in both directions from its axis. The stresses in the members are transferred by means of cover plates riveted on both sides of each channel, and not through the pin, which merely connects the upper chord with the web members.

Fig. 524 is an elevation of the connection at upper end of 7 8, and Fig. 525 is a cross-section near the pin. Fig. 526 is the diagram for bending moments acting on pin perpendicular to axis of 7 8; Fig. 527 is a similar diagram for moments acting in the same plane as the axis of 7 8. The intercepts found are combined in Fig. 528 to obtain the maximum intercept in inches, which is multiplied by pole distance in tons to obtain numerical value of maximum bending moment in inch-tons.

Then $1\frac{13}{16}''$ = diameter of pin to resist shearing. (Table U.)

$3\frac{1}{16}''$ = diameter to resist bearing.

$2\frac{9}{16}''$ = diameter to resist maximum bending moment.

Hence the pin is made $3\frac{1}{16}''$ in diameter. The dimensions of pin ends and pin nuts are given in Cambria, page 344, and in Carnegie, page 200.

The member 7 8 is here composed of two Ls, between which is riveted a gusset end plate, bored to slip on the pin. Members 6 7 and 8 9 each consist of two rods with loop-welded eyes (Cambria, page 346).

Figs. 529 and 530 are the elevation and section of the connection at lower end of member 7 8. Each member of the lower chord consists of two flat eye-bars of rectangular section, and 8 15 is composed of two rods with loop-welded eyes.

Then $1\frac{11}{16}''$ = diameter of pin to resist shearing.

$2\frac{7}{16}''$ = diameter to resist bearing.

$2\frac{1}{2}''$ = diameter to resist maximum bending moment.

Hence the pin is made $2\frac{9}{16}''$ as $2\frac{1}{2}''$ is not a regular size of pin.

501. Rivet Connections.—Fig. 531 shows a riveted connection at the upper end of 7 8. The upper chord here consists of two Ls connected by top and cover plates, while the gusset connects all members together. The web members are each composed of two Ls with both flanges riveted to the gusset, except for 7 8 where this is not necessary. Cross-sections of the different members are given.

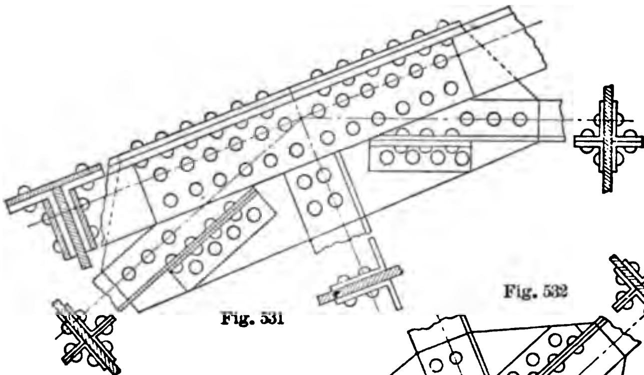


Fig. 531

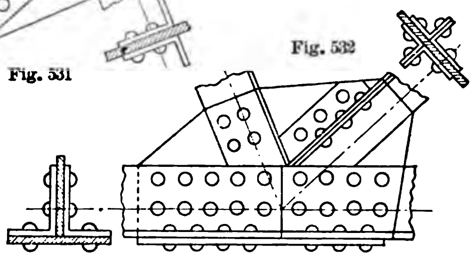


Fig. 532

Fig. 532 is the riveted connection at the lower end of 7 8 joining the lower chord and web members.

By comparing these pin and riveted connections, it is at once apparent that the latter is more easily designed and more economically constructed; therefore riveted connections are now generally employed, excepting for roof trusses of very wide spans or to be erected in a distant country, where expert riveters may not be obtained. Bending moments do occur in riveted connections, but these are usually small enough to be neglected.

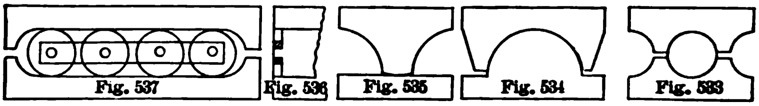
502. Slip Plates (Fig. 535).—The ends of roof trusses generally have horizontal steel or cast-iron plates, that rest on the tops of the walls and are fastened by anchor bolts. This suffices for moderate spans. But wider spans require provision for expansion and contraction of the length of the truss in order to prevent injury to the walls. This may be provided by a slip plate at one end, as in Fig. lower 535, the upper plate being fixed to the truss and sliding on the

one that is fixed to the wall. Its bearing surface should be slightl cylindrical as shown.

Let r = radius of curvature in inches of the bearing surface.

l = its length in inches.

Then $0.75\ r\ l$ = maximum safe load in tons transmitted. . (168



503. Rockers (Figs. 533, 534).—In Fig. 533 a cylindrical stee pin is set between the plates, which are accurately fitted to it; i may then be regarded as a pin subject to a bearing stress.

Then for a steel pin between steel plates:

$12.5\ d\ l$ = maximum safe load in tons transmitted. . . (169)

But for a steel pin between cast-iron plates:

$5.0\ d\ l$ = maximum safe load in tons transmitted. . . (170)

It is therefore necessary to use steel plates for heavy loads. The same form of connection is also employed at ridge apex of a jointed arch truss.

Fig. 534 shows a cylindrical bearing surface on the lower plate both plates being assumed to be of cast iron.

Then $5\ d\ l$ = maximum safe load in tons transmitted. . (171)

504. Expansion Rolls (Figs. 536, 537).—Cylindrical steel rolls are inserted between flat plates to permit movement of the uppe plate caused by expansion or contraction of the truss. These rolls are kept parallel and perpendicular to the plane of the truss by steel bars fastened on their projecting ends.

Let n = number of rolls.

l = length in inches of one roll.

d = diameter in inches of a roll.

Then $0.15\ n\ l\ d$ = maximum safe load in tons transmitted. . (172)

CHAPTER X

TABLES FOR DIMENSIONING MEMBERS

505. Purpose of the Tables.—The necessary computations may be made by using the proper formulas previously given in Chapter IX. But a series of original numerical and graphical tables have been constructed by the author for economizing time and labor, as well as the avoidance of errors. These tables generally give the desired dimensions by simple inspection. Some have been tested by years of actual use; others have been recently devised; all have been adapted to the coefficients and data now generally accepted in the United States.

I. TENSION

506. Table A.—Rods without upset ends, with nuts and washers. Since the safe tensile strength of the rod is reduced nearly 30 per cent by cutting screw threads at each end to receive the nuts, this table should always be used, unless it is absolutely certain that the rod ends are to be enlarged to the standard upset ends, which have a safe resistance not less than that of the rod itself.

Safe tensile strength allowed is 8 tons per sq. in. of reduced net sectional area of the rod.

Dimensions of nuts are manufacturers' standard sizes, which are now generally employed instead of the Franklin Institute standards, because these nuts are cut from standard bars without waste.

Washers are made of cast iron, have the same thickness as the diameter of the rod, with hole $\frac{1}{16}$ in. larger than the rod. They are usually tapered in thickness toward the edges, which should be $\frac{3}{8}$ to $\frac{3}{4}$ inch.

The net safe areas of washers are given in sq. ins. for woods with values of $C' = .30, .25, .20, .175, .15, .125, .10,$ and $.075$ ton per sq. inch against side of timber. Areas required for intermediate values of C' can be easily interpolated. To the net area of washer is to be added the area of hole for rod in order to obtain total required area of washer.

When the rod exceeds 2 ins. diameter, it is better to use a group of two or three rods of equal resistance, to compose the member of the roof truss.

TABLE A

STEEL RODS—ENDS NOT UPSET

Net area C. I. Washer in sq. ins

Diam.	Safe Tension	Nut Square	Values of C′							
			.300	.250	.200	.175	.150	.125	.100	.075
In.	Tons		Sq. in.							Sq. in.
½	1.01	½× ⅞	3.4	4.0	5.0	5.8	6.7	8.1	10.1	13.4
9/16	1.30	9/16×1⅛	4.4	5.2	6.5	7.5	8.7	10.4	13.0	17.4
⅝	1.62	⅝×1⅛	5.4	6.5	8.1	9.2	10.8	12.9	16.2	21.5
11/16	1.96	11/16×1¼	6.5	7.8	9.8	11.2	13.1	15.7	19.6	26.1
¾	2.42	¾×1⅜	8.1	9.7	12.1	13.8	16.1	19.3	24.2	32.2
13/16	2.80	13/16×1⅜	9.3	11.2	14.0	16.0	18.7	22.4	28.0	37.1
⅞	3.36	⅞×1⅝	11.2	13.4	16.8	19.2	22.4	26.9	33.6	44.8
15/16	3.86	15/16×1⅝	12.9	15.5	19.3	22.1	25.8	30.9	38.6	51.5
1	4.50	1 ×1¾	15.0	18.0	22.5	25.7	30.0	36.0	45.0	57.9
1 1/16	5.19	1 1/16×1¾	17.3	20.8	26.0	29.7	34.6	41.5	51.9	69.2
1⅛	5.55	1⅛×2	18.5	22.2	27.8	31.8	37.0	44.4	55.5	74.0
1 3/16	6.33	1 3/16×2	21.1	25.3	31.6	36.2	42.2	50.6	63.3	84.3
1¼	7.13	1¼×2¼	23.8	28.5	35.6	40.8	47.5	57.0	71.3	95.0
1 5/16	7.99	1 5/16×2½	26.6	32.0	40.0	45.7	53.3	63.9	79.9	106.5
1⅜	8.44	1⅜×2¾	28.1	33.8	42.2	48.3	56.3	67.5	84.4	112.5
1 7/16	9.39	1 7/16×2¾	31.3	37.5	46.9	53.7	62.6	75.1	93.8	125.0
1½	10.36	1½×3	34.5	41.4	51.8	59.3	69.1	82.9	103.6	138.1
1 9/16	11.40	1 9/16×3	38.0	45.6	57.0	65.2	76.0	91.2	114.0	151.9
1⅝	12.14	1⅝×3¼	40.5	48.5	60.7	69.4	80.9	97.1	121.4	156.7
1 11/16	13.26	1 11/16×3¼	44.2	53.1	66.3	75.9	88.5	106.1	132.6	176.7
1¾	13.97	1¾×3½	46.6	55.9	69.8	79.9	93.2	111.7	139.7	186.1
1 13/16	15.18	1 13/16×3½	50.6	60.7	75.9	86.8	101.2	121.4	151.8	202.2
1⅞	16.41	1⅞×3¾	54.7	65.6	82.0	93.8	109.4	131.3	164.1	218.6
1 15/16	17.71	1 15/16×3¾	59.1	70.9	88.6	101.3	118.1	141.7	177.1	236.0
2	18.44	2 ×4	61.5	73.8	92.2	105.5	123.0	147.5	184.4	245.7
2⅛	21.23	2⅛×4	70.8	84.9	106.2	121.4	141.6	169.9	212.3	282.9
2¼	24.22	2¼×4¼	80 7	96.9	121.1	138.5	161.5	193.7	242.2	322.7
2⅜	26.41	2⅜×4¼	88.0	105.6	132.0	151.0	176.1	211.3	264.1	351.9
2½	29.73	2½×4½	99.1	118.9	148.6	170.0	198.3	237.8	297.3	396.1
2⅝	33.24	2⅝×4½	110.8	133.0	166.2	190.1	221.7	265.9	332.4	442.9
2¾	36.95	2¾×4¾	123.2	147.8	184.8	211.3	246.4	295.6	369.5	492.4
2⅞	40.86	2⅞×5	136.2	163.4	204.3	233.7	272.5	326.9	408.6	544.4
3	43.40	3 ×5	144.7	173.6	217.0	248.2	289.4	347.2	434.0	578.3

507. Example.—25 tons = tensile stress in the member. Longleaf pine.

It is best to use two rods with 12.5 tons tensile stress in each.

By Table A, 2 rods $1\frac{11}{16}''$ diameter; nut $1\frac{3}{4} \times 3\frac{1}{2}$ sq., C' for longleaf pine = .175; hence total area of washer = $2 \times 75.9 + 2 \times 2.41 = 156.6$ sq. ins. $\times\ 1\frac{11}{16}''$ thick, tapered to $\frac{3}{4}$ inch at edge, with 2, $1\frac{3}{4}''$ holes for rods. If set on a timber 12 ins. wide, its length = $\frac{156.6}{12} = 13.5$ ins.

508. Table B.—Rods with standard upset ends, nuts, and cast-iron washers.

For dimensions of standard upset ends, see Cambria, pages 384, 385; Carnegie, pages 205, 206. The use of these rods with upset ends produces considerable economy in weight, but the upsets must be made with standard dies, never by welding on a piece of a larger rod.

Diameter of upset end, thickness of nut, and thickness of washer are all made equal to each other. Net areas of washers are given for values of C' = .30, .25, .20, .175, .15, .125, .10, and .075, as in Table A.

509. Example.—25 tons = tensile stress in member as before. Longleaf pine.

By Table B, 2 rods $1\frac{7}{16}''$ diameter will suffice; upset end $1\frac{7}{8}'' \times 5\frac{1}{2}''$ long; nut $1\frac{7}{8}'' \times 3\frac{3}{4}''$ square; total area of washer $= 2 \times 74.3 + 2 \times 2.95 = 154.5$ sq. ins.; thickness $1\frac{7}{8}''$, tapered to $\frac{3}{4}''$ at edge; with two holes $1\frac{15}{16}''$ diameter for rods. If set on $12''$ timber, then its length $= \frac{154.5}{12} = 12.9$ ins., a little less than in previous example.

510. Table C.—Two steel channels in tension, with webs only riveted.

TABLE B

STEEL RODS—ENDS UPSET

Net area of C. I. Washer

Diam.	Safe Tension	Upset End	Nut	Drop	Values of C′							
					.300	.250	.200	.175	.150	.125	.100	.075
In.	Tons	In.	In.	In.	Sq. in.							Sq. in.
½	1.57	¾×4¼	¾×1⅜	2½	5.2	6.3	7.9	9.0	10.5	12.6	15.7	20.9
9/16	1.99	¾×4¼	¾×1⅜	2½	6.6	8.0	9.9	11.4	13.3	15.9	19.9	26.5
⅝	2.46	⅞×4½	⅞×1½	2¼	8.2	9.8	12.3	14.0	16.4	19.6	24.5	32.7
11/16	2.97	1 ×4½	1 ×1¾	2	9.9	11.9	14.9	17.0	19.8	23.8	29.7	39.6
¾	3.54	1 ×4½	1 ×1¾	2	11.8	14.1	17.7	20.2	23.6	28.3	35.3	47.1
13/16	4.14	1⅛×4¾	1⅛×2	2	13.8	16.6	20.7	23.7	27.7	33.2	41.5	55.3
⅞	4.81	1¼×4¾	1¼×2¼	1¾	16.0	19.2	24.1	27.5	32.1	38.5	48.1	64.1
15/16	5.52	1¼×4¾	1¼×2¼	1¾	18.4	22.1	27.6	31.6	36.8	44.2	55.2	73.6
1	6.28	1⅜×5	1⅜×2¾	1¾	21.0	25.1	31.4	35.9	41.9	50.3	62.8	83.7
1 1/16	7.10	1⅜×5	1⅜×2¾	1¾	23.7	28.4	35.5	40.6	47.3	56.7	70.9	94.5
1⅛	7.95	1½×5	1½×3	1½	26.5	31.8	39.8	45.5	53.0	63.6	79.5	106.0
1 3/16	8.87	1½×5	1½×3	1½	29.5	35.4	44.3	50.7	59.1	70.9	88.6	118.1
1¼	9.82	1⅝×5¼	1⅝×3¼	1½	32.7	39.3	49.1	56.1	65.5	78.5	98.2	130.8
1 5/16	10.83	1¾×5¼	1¾×3½	1¼	36.1	43.3	54.1	61.9	72.2	86.6	108.2	144.2
1⅜	11.68	1¾×5¼	1¾×3½	1¼	39.6	47.5	59.4	67.9	79.2	95.0	118.8	158.3
1 7/16	12.99	1⅞×5½	1⅞×3¾	1¼	43.3	51.9	64.9	74.3	86.6	103.9	129.8	173.0
1½	14.14	2 ×5½	2 ×4	1	47.1	56.6	70.7	80.8	94.3	113.1	141.4	188.4
1 9/16	15.34	2 ×5½	2 ×4	1	51.1	61.4	76.7	87.7	102.3	122.7	153.4	204.4
1⅝	16.59	2⅛×5¾	2⅛×4	1	55.3	66.4	83.0	94.9	110.6	132.7	165.9	221.1
1 11/16	17.90	2⅛×5¾	2⅛×4	1	59.7	71.6	89.5	102.3	119.3	143.1	178.9	238.4
1¾	19.24	2¼×5¾	2¼×4¼	¾	64.2	77.0	96.2	110.0	128.3	153.9	192.4	256.4
1 13/16	20.64	2¼×5¾	2¼×4¼	¾	68.8	82.6	103.2	118.0	137.7	165.1	206.4	275.1
1⅞	22.09	2⅜×6	2⅜×4¼	¾	73.6	88.4	110.5	126.3	147.3	176.7	220.9	294.3
1 15/16	23.58	2½×6	2½×4½	½	78.6	94.4	117.9	134.9	157.3	188.7	235.9	314.3
2	25.14	2½×6	2½×4½	½	83.8	100.5	125.7	143.7	167.6	201.1	251.3	334.9
2⅛	28.38	2⅝×6¼	2⅝×4½	½	94.6	113.5	141.9	162.3	189.2	227.0	283.7	378.1
2¼	31.81	2⅞×6½	2⅞×5	¼	106.0	127.2	159.0	181.9	212.1	254.5	318.1	423.9
2⅜	35.44	3 ×6½	3 ×5¼	0	118.2	141.8	177.2	202.7	236.4	283.5	354.4	472.3
2½	39.27	3⅛×6¾	3⅛×5½	0	130.9	157.1	196.4	224.6	261.9	314.2	392.7	523.3
2⅝	43.30	3¼×6¾	3¼×5¾	−¼	144.3	173.2	216.5	247.6	288.7	346.4	433.0	576.9
2¾	47.52	3⅜×7	3⅜×6	−¼	158.4	190.1	237.6	271.7	316.9	380.1	475.2	632.2
2⅞	51.94	3⅝×7¼	3⅝×6¼	−½	173.1	207.7	259.7	297.0	346.3	415.5	519.3	692.0
3	54.22	3¾×7½	3¾×6½	−½	188.5	226.2	282.7	323.4	377.1	452.4	565.5	753.5

Deduction from total sectional area of webs only must be made for all rivet holes in them, taking diameter of hole $\frac{1}{8}''$ larger than

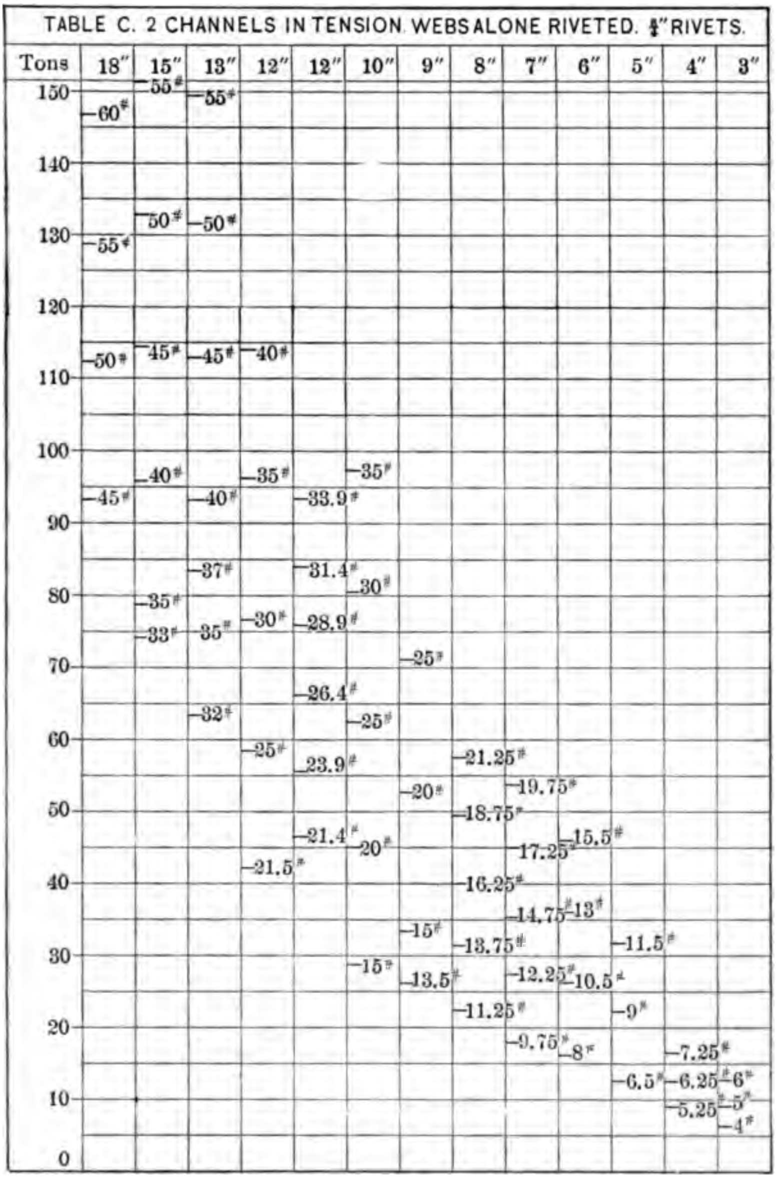


TABLE C. 2 CHANNELS IN TENSION. WEBS ALONE RIVETED. ¾" RIVETS.
Tons
18" 15" 13" 12" 12" 10" 9" 8" 7" 6" 5" 4" 3"
150 140 130 120 110 100 90 80 70 60 50 40 30 20 10 0
60# 55# 50# 45#
55# 50# 45# 40# 35# 33#
55# 50# 45# 40# 37# 35# 32#
40# 35# 30# 25# 21.5#
33.9# 31.4# 28.9# 26.4# 23.9# 21.4#
35# 30# 25# 20# 15#
25# 20# 15# 13.5#
21.25# 18.75# 16.25# 13.75# 11.25#
19.75# 17.25# 14.75# 12.25# 9.75#
15.5# 13# 10.5# 8#
11.5# 9# 6.5#
7.25# 6.25# 5.25#
6# 5# 4#

diameter of rivet, to allow for injury to metal by punching rivet hole. The arrangement of holes punched in webs is according to standards of American Bridge Co. Maximum safe tensile stress of 8 tons per square inch of least net area of cross-section of webs only, is allowed. This tensile stress is here assumed to be resisted by the webs alone, since the flanges are not connected by rivets and cover plates. Fig. 520.

The table gives the maximum safe tensile stress in tons for two channels from 3″ to 18″ depth, in the standard weights.

511. Example.—70 tons tensile stress in the member.

By Table C, 2, 9″25 # channels would suffice and would be lightest.

512. Table D.—Two steel channels in tension, riveted through both webs and flanges. Standard punching of American Bridge Co.

Deduction for area of rivet holes in any cross-section is made, taking diameter of hole $\frac{1}{8}''$ larger than diameter of rivet, as before. This construction is preferable and more economical for spliced channels in tension, since the flanges are made to resist their part of the stress.

513. Example.—70 tons tensile stress as before.

By Table D, 2, 8″ $21\frac{1}{4}$ # channnels would suffice and should be used.

514. Table E.—Two steel angles in tension, with wide legs alone riveted, $\frac{3}{4}''$ rivets.

The tensile stress is here assumed to be borne by the wide flange only, omitting the projecting portion of the narrow leg. Standard punching and deduction for rivet holes as before. Special angles are indicated by a star and should be avoided, except for a large order and extended time, or when found in stock.

The upper figures give the maximum safe tensile resistance in tons.

The lower figures give the weight of the two angles in lbs. per foot.

515. Example.—Tensile stress of 35 tons acting in the member.

By the table several different sections are found to suffice; 2, $6'' \times 4 \times \frac{9}{16}''$, 36.2 #; 2, $6 \times 3\frac{1}{2} \times \frac{9}{16}''$, 34.2 #; 2, $5 \times 3\frac{1}{2} \times \frac{11}{16}''$,

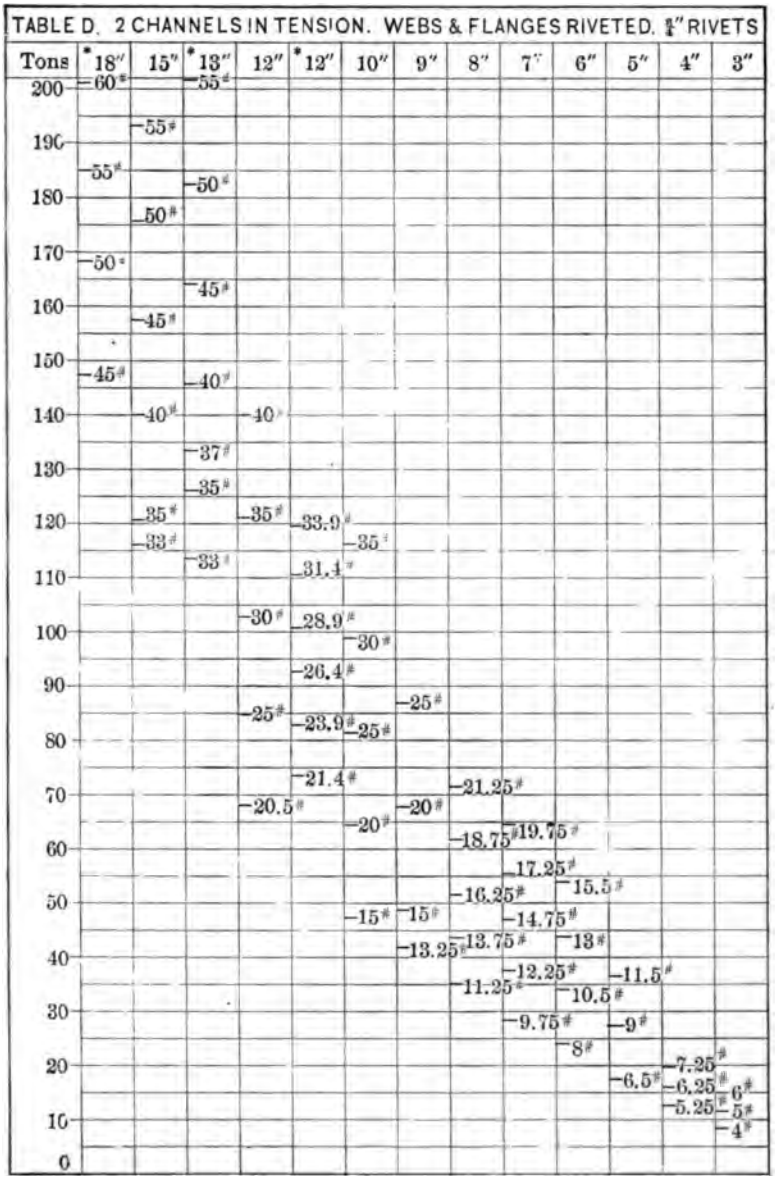


TABLE D. 2 CHANNELS IN TENSION. WEBS & FLANGES RIVETED. ¾" RIVETS
Tons
*18"
15"
*13"
12"
*12"
10"
9"
8"
7"
6"
5"
4"
3"
200
190
180
170
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
60#
55#
50#
45#
55#
50#
45#
40#
35#
33#
55#
50#
45#
40#
37#
35#
33#
40#
35#
30#
25#
20.5#
33.9#
31.4#
28.9#
26.4#
23.9#
21.4#
35#
30#
25#
20#
15#
25#
20#
15#
13.25#
21.25#
18.75#
16.25#
13.75#
11.25#
19.75#
17.25#
14.75#
12.25#
9.75#
15.5#
13#
10.5#
8#
11.5#
9#
6.5#
7.25#
6.25#
5.25#
6#
5#
4#

TABLE E

TWO ANGLES IN TENSION—WIDE FLANGE RIVETED—THREE-FOURTH INCH RIVETS

Two Angles	Thickness of Angle															
	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8
8 × 8	....	Safe tension, tons Weight, 2 angles, lbs. per ft.				50.0 52.8	56.3 59.2	62.5 65.4	68.7 71.6	75.0 77.8	81.3 84.0	87.5 90.0	93.8 96.2	100.0 102.0	106.3 108.0	120.6T 113.8#
7 × 3½*	......	......	......	......	36.8 30.0	42.0 34.0	46.2 38.2	52.5 42.0	57.8 46.0	63.0 49.8	68.3 53.6	73.5 57.4	78.8 61.0	84.0 64.6	Tons lbs. per ft.	
6 × 6	......	......	......	25.5 29.8	29.8 34.4	34.0 39.2	38.3 43.8	42.5 48.4	46.7 53.0	51.0 57.4	55.3 62.0	59.6 66.2	63.8 70.6	68.0 74.8		
6 × 4	......	......	......	25.5 24.6	29.8 28.6	34.0 32.4	38.3 36.2	42.5 40.0	46.7 43.6	51.0 47.2	55.3 50.8	59.6 54.4	63.8 57.8	68.0 61.2		
6 × 3½	......	......	......	25.5 23.4	29.8 27.0	34.0 30.6	38.2 34.2	42.5 37.8	46.7 41.2	51.0 44.8	55.3 48.0	59.6 51.4	63.8 54.6	68.0 57.8		
5 × 5*	......	......	......	19.5 24.6	22.8 28.6	26.0 32.4	29.4 36.2	32.5 40.0	35.8 43.6							
5 × 4*	......	......	......	19.5 22.0	22.8 25.6	26.0 29.0	29.4 32.4	32.5 35.6	35.8 39.0							
5 × 3½	......	......	16.3 17.4	19.5 20.8	22.8 24.0	26.0 27.2	29.4 30.4	32.5 33.6	35.8 36.6	39.0 39.6	42.3 42.6	45.5 45.4	48.8 48.4			
5 × 3	......	......	16.3 16.4	19.5 19.6	22.8 22.6	26.0 25.6	29.4 28.6	32.5 31.4	35.8 34.2	39.0 37.0	42.3 39.8	45.5 42.4				
4½ × 4½*	......	......	18.1 18.6	22.1 22.0	25.4 25.6	29.0 29.0	32.6 32.4	36.2 35.6	39.8 39.0							
4½ × 3*	......	......	......	22.1 18.2	25.4 21.2	29.0 23.8	32.6 26.6	36.2 29.4	39.8 32.0							
4 × 4	......	......	15.6 16.4	18.7 19.6	21.8 22.6	25.0 25.6	28.1 28.6	31.2 31.4	34.3 34.2	37.4 37.0	40.6 39.8	44.1 42.4				

Two Angles	Thickness of Flanges $\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{13}{16}$	$\frac{7}{8}$	$\frac{15}{16}$	1	$1\frac{1}{16}$	$1\frac{1}{8}$
4 × 3½*	......	......	15.6	18.7	21.8	25.0	28.1	31.2	34.3							
			15.4	18.2	21.2	23.8	26.6	29.4	32.0							
4 × 3	......	......	15.6	18.7	21.8	25.0	28.1	31.2	34.3	37.4	40.6	43.8				
			14.4	17.0	19.6	22.2	24.8	27.2	29.6	32.0	34.2	36.6				
3½ × 3½	......	......	13.1	15.8	18.4	21.0	23.8	26.3	28.9	31.5	34.2	36.8				
			14.4	17.0	19.6	22.2	24.8	27.2	29.6	32.0	34.2	36.6				
3½ × 3	......	......	13.1	15.8	18.4	21.0	23.8	26.3	28.9	31.5	34.2	36.8				
			13.2	15.8	18.2	20.4	22.8	25.0	27.2	29.4	31.6	33.6				
3½ × 2½	......	10.5	13.1	15.8	18.4	21.0	23.8	26.3	28.9	31.5						
		9.8	12.2	14.4	16.6	18.8	20.8	23.0	25.0	26.8						
3½ × 2*	......	10.5	13.1	15.8	18.4	21.0	23.8	26.3								
		9.0	11.2	13.2	15.2	17.0	19.0	20.8								
3 × 3	......	8.5	10.6	12.7	14.9	17.0	19.1	21.3	23.4							
		9.8	12.2	14.4	16.6	18.8	20.8	23.0	25.0							
3 × 2½	......	8.5	10.6	12.7	14.9	17.0	19.1	21.3								
		9.0	11.2	13.2	15.2	17.0	19.0	20.8								
3 × 2*	6.3	8.5	10.6	12.7	14.8	17.0										
	6.2	8.2	10.0	11.8	13.6	15.4										
2¾ × 2¾*	5.6	7.4	9.4	11.7	13.0	15.0										
	6.8	9.0	11.2	13.2	15.2	17.0										
2½ × 2½	5.0	6.5	8.2	9.8	11.4	13.0	14.7									
	6.2	8.2	10.0	11.8	13.6	15.4	17.0									
2½ × 2	4.9	6.5	8.1	9.8	11.4	13.0	14.6									
	5.6	7.4	9.0	10.6	12.2	13.6	15.2									
2¼ × 2¼*	4.1	5.4	6.9	8.2	9.6											
	5.6	7.4	9.0	10.6	12.2											
2 × 2	3.4	4.5	5.7	6.7	7.8	9.0										
	5.0	6.4	8.0	9.4	10.6	12.0										

TABLE F

TWO ANGLES IN TENSION—WIDE FLANGE RIVETED—SEVEN-EIGHTH INCH RIVETS

Two Angles.	Thickness of Angle															
	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8
8×8	.....	Safe tension, tons				48.0	54.0	60.0	66.0	72.0	78.0	84.0	90.0	96.0	102.0	108.0
		Weight, 2 angles, lbs. per ft.				52.8	59.2	65.4	71.6	77.8	84.0	90.0	96.2	102.0	108.0	113.8
7×3½*					35.0	40.0	45.0	50.0	55.0	60.0	65.0	70.0	75.0	80.0	Tons.	
					30.0	34.0	38.2	42.0	46.0	49.8	53.6	57.4	61.0	64.6	lbs. per ft.	
6×6				24.0	28.0	32.0	36.0	40.0	44.0	48.0	52.0	56.0	60.0	64.0		
				29.8	34.4	39.2	43.8	48.4	53.0	57.4	62.0	66.2	70.6	74.8		
6×4				24.0	28.0	32.0	36.0	40.0	44.0	48.0	52.0	56.0	60.0	64.0		
				24.6	28.6	32.4	36.2	40.0	43.0	47.2	50.8	54.4	57.8	61.2		
6×3½				24.0	28.0	32.0	36.0	40.0	44.0	48.0	52.0	56.0	60.0	64.0		
				23.4	27.0	30.6	34.2	37.8	41.2	44.8	48.0	51.4	54.6	57.8		
5×5*				18.0	21.0	24.0	27.0	30.0	33.0							
				24.6	28.6	32.4	36.2	40.0	43.6							
5×4*				18.0	21.0	24.0	27.0	30.0	33.0							
				22.0	25.6	29.0	32.0	35.6	39.0							
5×3½			15.0	18.0	21.0	24.0	27.0	30.0	33.0	36.0	39.0	42.0	45.0			
			17.4	20.8	24.0	27.2	30.4	33.6	36.6	39.6	42.6	45.4	48.4			
5×3			15.0	18.0	21.0	24.0	27.0	30.0	33.0	36.0	39.0	42.0				
			16.4	19.6	22.6	25.6	28.6	31.4	34.2	37.0	39.8	42.4				
4½×4½*			17.4	21.0	24.5	28.0	31.5	35.0	38.5							
			18.6	22.0	25.6	29.0	32.4	35.6	39.0							
4½×3*				21.0	24.5	28.0	31.5	35.0	38.5							
				18.2	21.2	23.8	26.6	29.4	32.0							
4×4			15.0	18.0	21.0	24.0	27.0	30.0	33.0	36.0	39.0	42.0				
			16.4	19.6	22.6	25.6	28.6	31.4	34.2	37.0	39.8	42.4				
4×3½*			15.0	18.0	21.0	24.0	27.0	30.0	33.0							
			15.4	18.2	21.2	23.8	26.6	29.4	32.0							

TABLE F—*Continued*

Two Angles	Thickness of Angles															
	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8
4 × 3	......	......	15.0 14.4	18.0 17.0	21.0 19.6	24.0 22.2	27.0 24.8	30.0 27.2	33.0 29.6	36.0 32.0	39.0 34.2	42.0 36.6				
3½ × 3½	......	......	12.5 14.4	15.0 17.0	17.5 19.6	20.0 22.2	22.5 24.8	25.0 27.2	27.5 29.6	30.0 30.0	32.5 32.2	35.0 36.6				
3½ × 3	......	......	12.5 12.2	15.0 15.8	17.5 18.2	20.0 20.4	22.5 22.8	25.0 25.0	27.5 27.2	30.0 29.4	32.5 31.6	35.0 33.6				
3½ × 2½	......	10.0 9.8	12.5 12.5	15.0 14.4	17.5 16.6	20.0 18.8	22.5 20.8	25.0 23.0	27.5 25.0	30.0 26.8						
3½ × 2*	......	10.0 9.0	12.5 11.2	15.0 13.2	17.5 15.2	20.0 17.0	22.5 19.0	25.0 20.8								
3 × 3	......	8.0 9.8	10.0 12.2	12.0 14.4	14.0 16.6	16.0 18.8	18.0 20.8	20.0 23.0	22.0 25.0							
3 × 2½	......	8.0 9.0	10.0 11.2	12.0 13.2	14.0 15.2	16.0 17.0	18.0 19.0	20.0 20.8								
3 × 2*	6.0 6.2	8.0 8.2	10.0 10.0	12.0 11.8	14.0 13.8	16.0 15.4										
2¾ × 2¾*	5.5 6.8	7.0 9.0	8.5 11.2	11.0 13.2	12.5 15.2	14.0 17.0										
2½ × 2½	4.5 6.2	6.0 8.2	8.5 10.0	9.0 11.8	10.5 13.6	12.0 15.4	13.5 17.0									
2½ × 2	4.5 5.6	6.0 7.4	8.5 9.0	9.0 10.6	10.5 12.2	12.0 13.6	13.5 15.2									
2¼ × 2¼*	3.7 5.6	5.0 7.4	6.2 9.0	7.5 10.6	8.7 12.2											
2 × 2	3.0 5.0	4.0 6.4	5.0 8.0	6.0 9.4	7.0 10.6	8.0 12.0										

TABLE G

TWO ANGLES IN TENSION—BOTH FLANGES RIVETED—THREE-FOURTH INCH RIVETS

Two Angles	Thickness of Flanges															
	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8
8×8	Safe tension, tons					96.0	107.5	118.7	130.0	141.0	151.9	162.9	173.6	184.0	194.6	227.8
	Weight, 2 angles, lbs. per ft.					52.8	59.2	65.4	71.6	77.8	84.0	90.0	96.2	102.0	108.0	113.8
7×3½*					52.2	59.0	65.9	72.6	79.1	85.6	91.9	98.2	104.2	110.0	Tons.	
					30.0	34.0	38.2	42.0	46.0	49.8	53.6	57.4	61.0	64.6	lbs. per ft.	
6×6				48.8	56.5	64.0	71.5	78.7	86.0	93.0	99.9	106.9	113.6	120.0		
				29.8	34.4	39.2	43.8	48.4	53.0	57.4	62.0	66.2	70.6	74.8		
6×4				42.0	48.6	55.0	61.3	67.5	73.7	79.5	85.4	91.1	96.6	102.0		
				24.6	28.6	32.4	36.2	40.0	43.6	47.2	50.8	54.4	57.8	61.2		
6×3½				39.1	45.1	51.0	56.9	62.6	68.1	73.6	78.8	84.1	89.1	94.0		
				23.4	27.0	30.6	34.2	37.8	41.2	44.8	48.0	51.4	54.6	57.8		
5×5*				36.8	42.6	48.0	53.4	59.0	64.1							
				24.6	28.6	32.4	36.2	40.0	43.6							
5×4*				36.1	41.6	47.0	52.4	57.6	62.6							
				22.0	25.6	29.0	32.4	35.6	39.0							
5×3½			28.2	33.0	38.1	43.0	47.9	52.6	57.2	61.6	65.8	70.2	74.2			
			17.4	20.8	24.0	27.2	30.4	33.6	36.6	39.6	42.6	45.4	48.4			
5×3			25.2	30.0	34.6	39.0	43.4	47.5	51.6	55.5	59.3	63.1				
			16.4	19.6	22.6	25.6	28.6	31.4	34.2	37.0	39.8	42.4				
4½×4½*			34.8	41.4	47.8	54.0	60.2	66.3	72.2							
			18.6	22.0	25.6	29.0	32.4	35.6	39.0							
4½×3*				32.4	37.2	42.0	46.8	51.3	55.8							
				18.2	21.2	23.8	26.6	29.4	32.0							
4×4			29.8	35.3	40.7	46.0	51.3	56.4	61.2	66.0	70.7	75.8				
			16.4	19.6	22.6	25.6	28.6	31.4	34.2	37.0	39.8	42.4				
4×3½*			27.3	32.4	37.2	42.0	46.8	51.3	55.8							
			15.4	18.2	21.2	23.8	26.6	29.4	32.0							

TABLE G—*Continued*

Two Angles	Thickness of Flanges															
	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8
4×3	......	......	24.7	29.4	33.8	38.0	42.3	46.3	50.2	54.0	57.8	61.3				
			14.4	17.0	19.6	22.2	24.8	27.2	29.6	32.0	34.2	36.6				
3½×3½	......	......	24.7	29.4	33.8	38.0	42.3	46.3	50.2	54.0	57.8	61.3				
			14.4	17.0	19.6	22.2	24.8	27.2	29.6	32.0	34.2	36.6				
3½×3	......	......	22.3	26.3	30.3	34.0	37.7	41.4	44.7	48.1	51.4	54.4				
			13.2	15.8	18.2	20.4	22.8	25.0	27.2	29.4	31.6	33.6				
3½×2½	......	16.0	19.8	23.3	26.8	30.0	33.2	36.2	39.3	42.0						
		9.8	12.2	14.4	16.6	18.8	20.8	23.0	25.0	26.8						
3½×2*	......	14.1	17.4	20.4	23.3	26.0	28.7	31.3								
		9.0	11.2	13.2	15.2	17.0	19.0	20.8								
3×3	......	16.0	19.8	23.3	26.8	30.0	33.2	36.2	39.3							
		9.8	12.2	14.4	16.6	18.8	20.8	23.0	25.0							
3×2½	......	14.1	17.4	20.4	23.3	26.0	28.7	31.3								
		9.0	11.2	13.2	15.2	17.0	19.0	20.8								
3×2*	9.3	12.0	14.8	17.4	19.8	22.0										
	6.2	8.2	10.0	11.8	13.6	15.4										
2¾×2¾*	10.7	14.1	17.4	20.4	23.1	26.0										
	6.8	9.0	11.2	13.2	15.2	17.0										
2½×2½	9.3	12.0	14.8	17.4	19.8	22.0	32.2									
	6.2	8.2	10.0	11.8	13.6	15.4	17.0									
2½×2	7.7	10.1	12.2	14.3	16.2	18.0	19.7									
	5.6	7.4	9.0	10.6	12.2	13.6	15.2									
2¼×2¼*	7.7	10.1	12.2	14.3	16.2											
	5.6	7.4	9.0	10.6	12.2											
2×2	6.2	8.0	9.8	11.3	12.7	14.0										
	5.0	6.4	8.0	9.4	10.6	12.0										

TABLE H
TWO ANGLES IN TENSION—BOTH FLANGES RIVETED—SEVEN-EIGHTH INCH RIVETS

Two Angles	HICKNESS OF ANGLES															
	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8
8×8	.....	Safe tension, tons Weight, 2 angles, lbs.				92.0 52.8	103.4 59.2	113.8 65.4	124.5 71.6	135.0 77.8	145.4 84.0	155.8 90.0	166.1 96.2	176.0 102.0	186.1 108.0	195.8 113.8
7×3½*	......	......	......	......	49.5 30.0	56.0 34.0	62.4 38.2	68.9 42.0	75.0 46.0	81.1 49.8	87.0 53.6	92.9 57.4	98.5 61.0	114.0 64.6		
6×6	......	......	......	45.8 29.8	53.0 34.4	60.0 39.2	67.0 43.8	73.8 48.4	80.5 53.0	87.0 57.4	93.4 62.0	99.8 66.2	106.1 70.6	112.0 74.8		
6×4	......	......	......	39.8 24.6	46.0 28.6	52.0 32.4	57.8 36.2	63.8 40.0	69.5 43.6	75.0 47.2	80.5 50.8	85.8 54.4	91.0 57.8	96.0 61.2		
6×3½	......	......	......	36.9 23.4	42.5 27.8	48.0 30.6	53.4 34.2	58.8 37.8	63.9 41.2	69.1 44.8	73.9 48.0	78.8 51.4	83.4 54.6	88.0 57.8		
5×5*	......	......	......	33.8 24.6	39.0 28.6	44.0 32.4	49.0 36.2	53.8 40.0	58.5 43.6							
5×4*	......	......	......	33.8 22.0	39.0 25.6	44.0 29.0	49.0 32.4	53.8 35.6	58.5 39.0							
5×3½	......	......	25.9 17.4	30.8 20.8	35.4 24.0	40.0 27.2	44.5 30.4	48.9 33.6	53.0 36.6	57.1 39.6	61.0 42.6	64.9 45.4	68.4 48.4			
5×3	......	......	23.5 16.4	27.8 19.6	31.9 22.6	36.0 25.6	40.0 28.6	43.8 31.4	47.4 34.2	51.0 37.0	54.4 39.8	57.8 42.4				
4½×4½*	......	......	33.5 18.6	39.8 22.0	46.0 25.6	52.0 29.0	58.0 32.4	63.8 35.6	69.5 39.0							
4½×3*	......	......	......	30.9 18.2	35.8 21.2	40.0 23.8	44.6 26.6	48.8 29.4	53.0 32.0							
4×4	......	......	28.6 16.4	33.8 19.6	39.0 22.6	44.0 25.6	49.0 28.6	53.9 31.4	58.5 34.2	63.0 37.0	67.4 39.8	71.8 42.4				
4×3½*	......	......	26.0 15.4	30.9 18.2	35.4 21.2	40.0 23.8	44.6 26.6	48.8 29.4	53.0 32.0							

TABLE H—*Continued*

Two Angles	Thickness of Angles: 3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	$1^1/_{16}$	$1^1/_8$
4×3	......	......	23.4 14.4	27.8 17.0	32.1 19.6	36.0 22.2	40.1 24.8	43.8 27.2	47.4 29.6	51.0 32.0	54.5 34.2	57.8 36.6				
3½×3½	......	......	23.4 14.4	27.8 17.0	32.1 19.6	36.0 22.2	40.1 24.8	43.8 27.2	47.4 29.6	51.0 32.0	54.5 34.2	57.8 36.6				
3½×3	......	......	21.0 12.2	24.8 15.8	28.6 18.2	32.0 20.4	35.4 22.8	38.9 25.0	42.0 27.2	45.1 29.4	48.1 31.6	50.9 33.6				
3½×2½	......	15.0 9.8	18.5 12.2	21.8 14.4	25.0 16.6	28.0 18.8	31.0 20.8	33.8 23.0	36.6 25.0	39.0 26.8						
3½×2*	......	13.1 9.0	16.1 11.2	18.9 13.2	21.5 15.2	24.0 17.0	26.5 19.0	28.8 20.8								
3×3	......	15.0 9.8	18.5 12.2	21.8 14.4	25.0 16.6	28.0 18.8	31.0 20.8	33.8 23.0	36.6 25.0							
3×2½	......	13.1 9.0	16.1 11.2	18.9 13.2	21.5 15.2	24.0 17.0	26.5 19.0	28.8 20.8								
3×2*	8.6 6.2	11.0 8.2	13.5 10.0	15.8 11.8	18.0 13.8	20.0 15.4										
2¾×2¾*	10.0 6.8	13.1 9.0	16.1 11.2	18.9 13.2	21.4 15.2	24.0 17.0										
2½×2½	8.6 6.2	11.0 8.2	13.5 10.0	15.8 11.8	18.0 13.6	20.0 15.4	22.0 17.0									
2½×2	7.0 5.6	9.1 7.4	11.0 9.0	12.8 10.6	14.5 12.2	16.0 13.6	17.5 15.2									
2¼×2¼	7.0 5.6	9.1 7.4	11.0 9.0	12.8 10.6	14.5 12.2											
2×2	5.5 5.0	7.0 6.4	8.6 8.0	9.8 9.4	11.0 10.6	12.0 12.0										

36.6 #; 2, 5 × 3 × $\frac{11}{16}$″, 34.2 #; 2, 4 × 3 × $\frac{3}{4}$″, 32 #; 2, 3$\frac{1}{2}$ × 3 × $\frac{7}{8}$″, 33.6 #. The lightest section would then be 2, 4 × 3 × $\frac{3}{4}$″, 32.0 #, but some other size might be in stock and more convenient, because delay would be avoided.

516. Table F.—Two steel angles in tension, wide legs alone riveted, 7/8″ rivets.

This table is similar to the last, but the wide leg is weakened by larger rivet holes; hence $\frac{3}{4}$″ rivets are usually preferable.

517. Example.—Tensile stress of 35 tons as before.

By Table, 2, 6 × 4 × $\frac{9}{16}$″, 36.2 #; 2, 6 × 3$\frac{1}{2}$ × $\frac{9}{16}$″; 2, 5 × 3$\frac{1}{2}$ × $\frac{3}{4}$″, 39.6 #; 2, 5 × 3 × $\frac{3}{4}$″, 37.0 #; 2, 4 × 3 × $\frac{3}{4}$″, 32.0 #: 2, 3$\frac{1}{2}$ × 3 × $\frac{7}{8}$″, 33.6 #; would suffice. The 2, 4 × 3 × $\frac{3}{4}$″, 32.0 # would be lightest, as in the last example.

518. Table G.—Two steel angles in tension, both legs riveted; $\frac{3}{4}$″ rivets. This is usually most economical, since the resistance of both legs is utilized. The table is applicable to angles with equal or unequal legs, and it is used like Tables E and F.

519. Example.—Tensile stress of 35 tons as before.

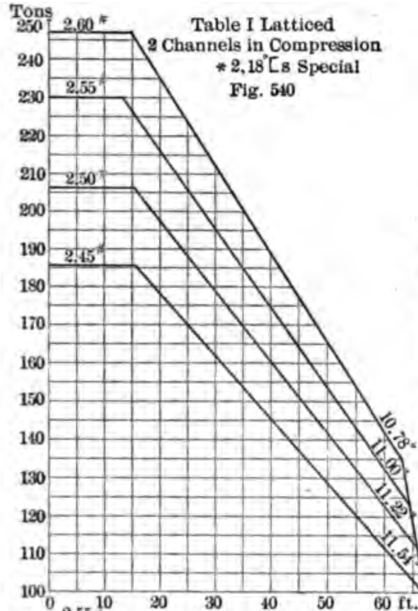
The table shows at once that 2, 4 × 3 × $\frac{3}{8}$″, 19.6 #, is the lightest section that could be used in this case.

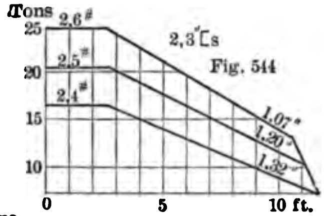
520. Table H.—Two steel angles in tension, both legs riveted; $\frac{7}{8}$″ rivets.

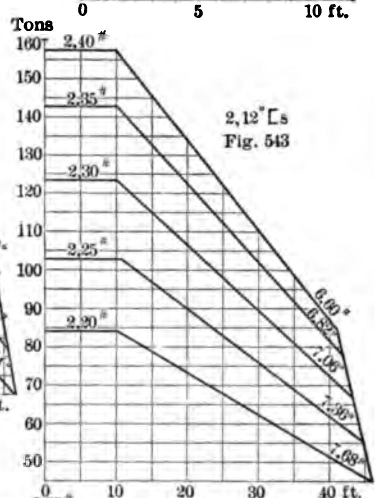
521. Example.—Tensile stress of 35 tons as before.

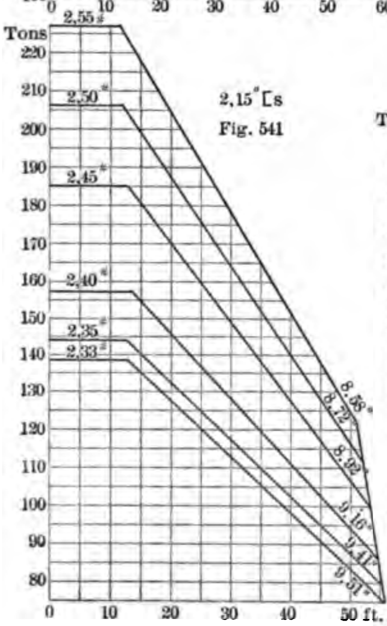
The table shows that 2, 4 × 3 × $\frac{1}{2}$″, 22.2 #, or 3$\frac{1}{2}$ × 3$\frac{1}{2}$ × $\frac{1}{2}$″, 22.2 #, would be equally light and either might be used.

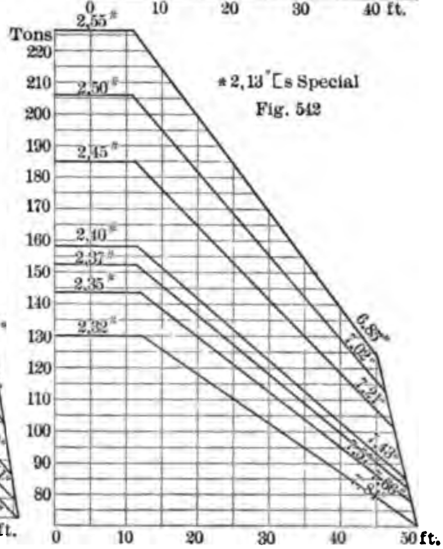
TABLE I—TWO LATTICED CHANNELS











II. COMPRESSION

Compressile stress lengthwise. Posts, struts, etc.

522. Tables I and J.—Two steel channels latticed together on flanges. The channels are spaced apart enough between webs to make the member equally stiff parallel or perpendicular to the webs. The minimum clear distance between webs is written just above the right-hand end of the inclined line representing the channel in the tables. Art. 492.

The maximum safe compression is limited to 7 tons per sq. in. of cross-section in accordance with the Chicago ordinance.

The channels are connected at end splices by web and flange cover plates, so as to distribute compressile stress over the entire cross-section. No deduction is made for rivet holes in channels. Top and bottom flanges are connected by lattice bars, but not by plates.

Table I contains diagrams for pairs of channels; 18″ special, 15″, 13″ special, 12″ and 3″. Table J comprises the diagrams for pairs of 10″, 9″, 8″, 7″, 6″, 5″, 4″, and 3″ channels.

At the left of each diagram is the maximum safe resistance in tons to compression; at the bottom is the length of channels in feet. A horizontal through the stress intersects a vertical through the length at a point on or just below the line representing the channel to be used. The maximum safe length in feet is here limited to 10 times the radius of gyration R° in inches, which is customary.

523. Example 1.—Strut 17 ft. long is subject to 35 tons compression. By Table J, 2, 6″, 13 #; 2, 7″, $12\frac{1}{4}$ #; 2, 8″, $11\frac{1}{4}$ #, would suffice.

The last pair are lightest and therefore most economical.

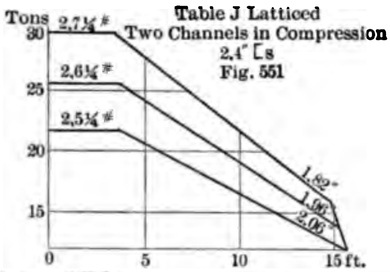
524. Example 2.—A column is composed of 2 latticed channels, 8″, $16\frac{1}{4}$ #, and it is required to safely support a vertical load of 50 tons. Required its maximum safe length.

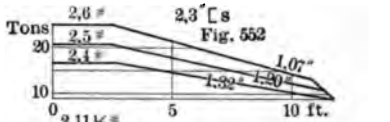
By Table J, this safe length = 19.0 feet.

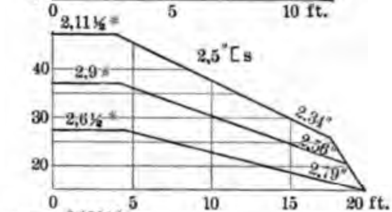
525. Tables K and L.—Two steel channels in compression are riveted to $\frac{3}{4}$″ gusset plate between them.

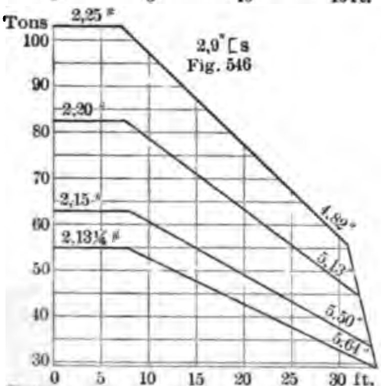
This is evidently not an economical construction, but it may sometimes be required because riveted connections are much simpler

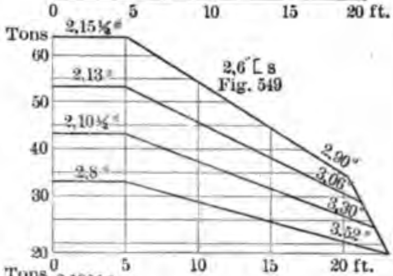
TABLE J—TWO LATTICED CHANNELS

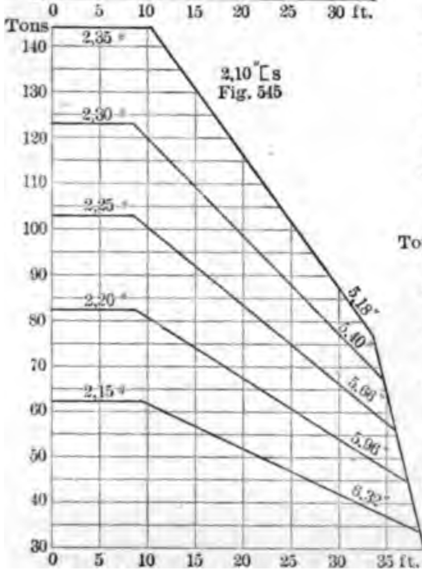


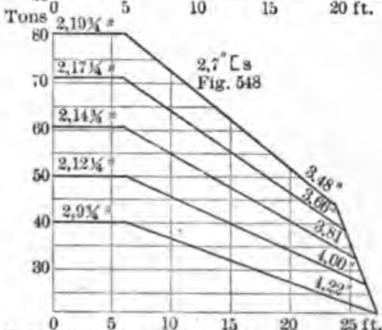












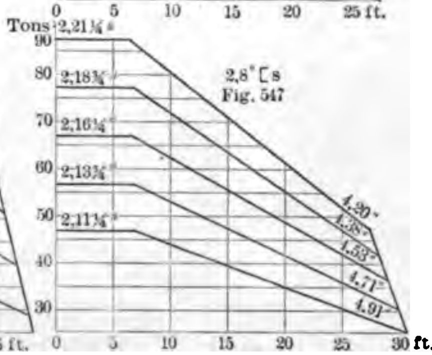


TABLE K—TWO CHANNELS WITH GUSSET

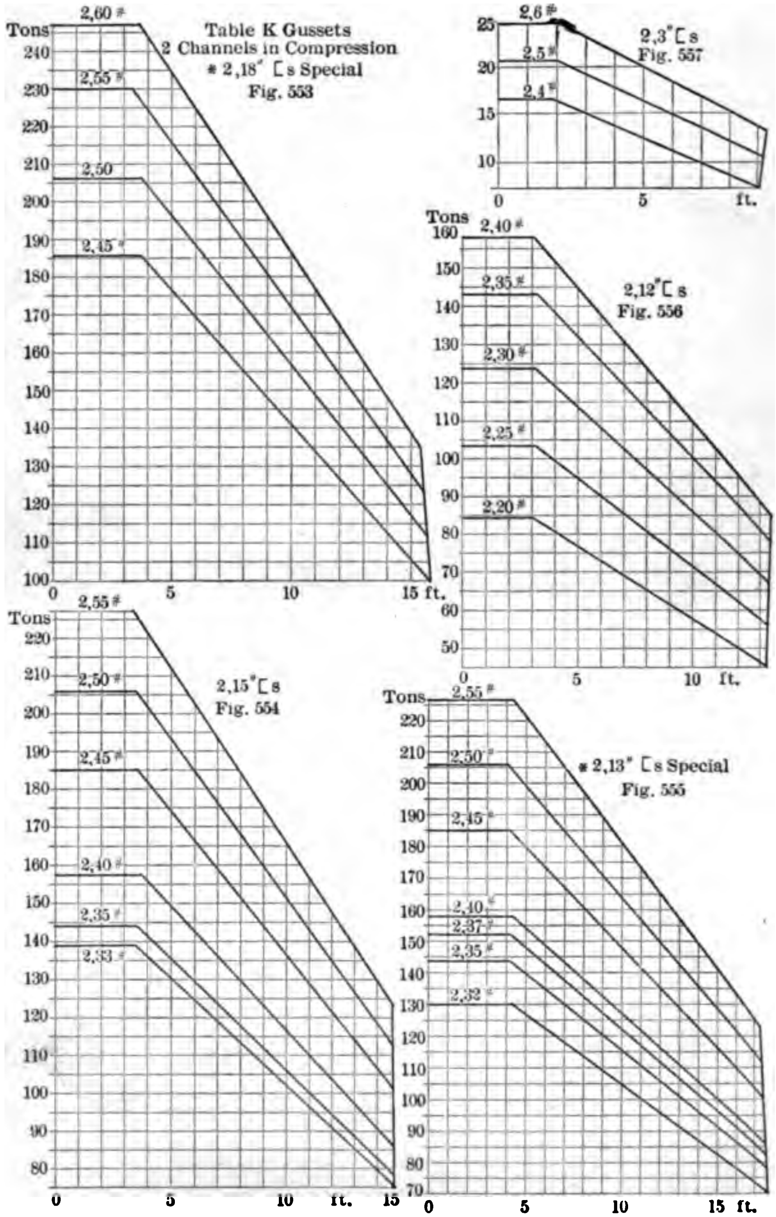
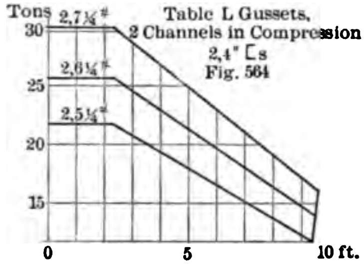
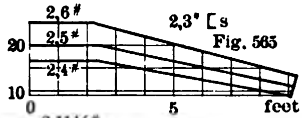
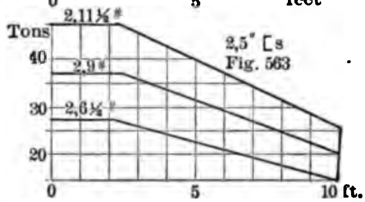
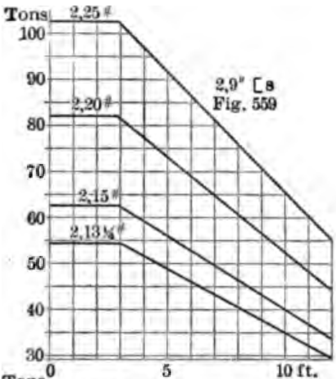


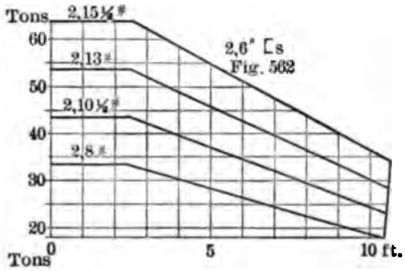
TABLE L—TWO CHANNELS WITH GUSSET

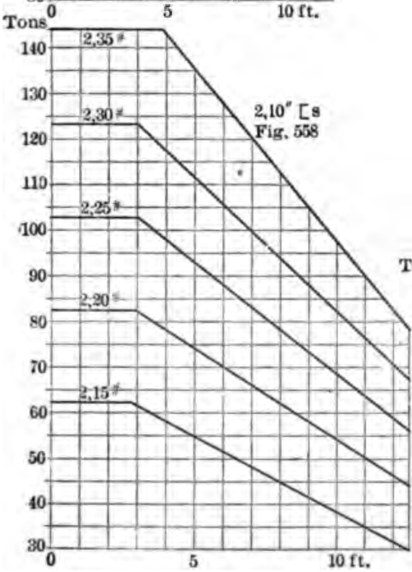


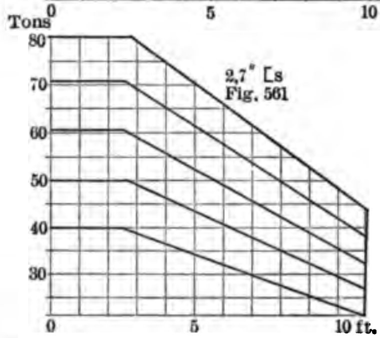












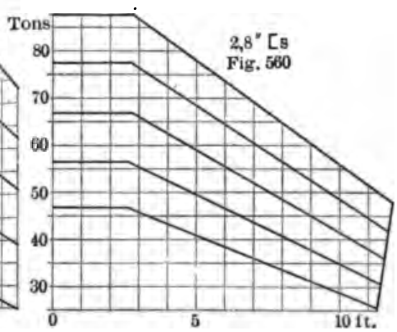


TABLE M—TWO ANGLES WITH GUSSET

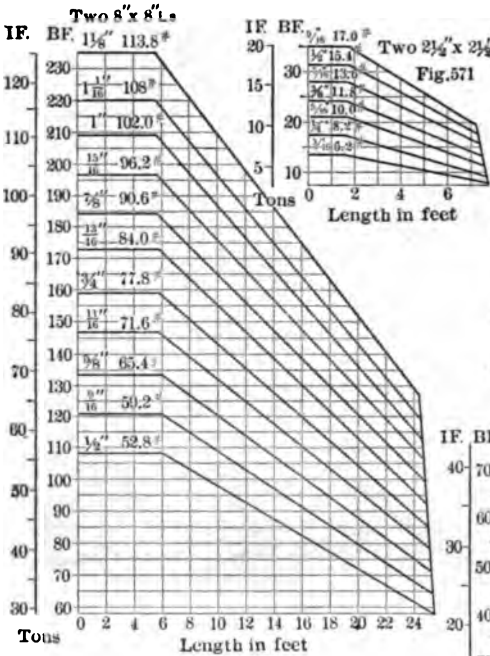


Fig.566

Fig.571

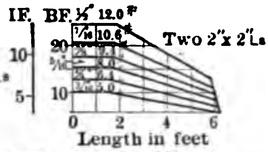


Fig.572

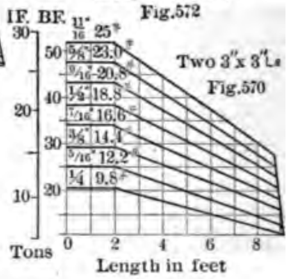


Fig.570

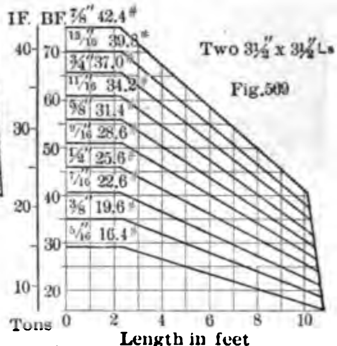


Fig.569

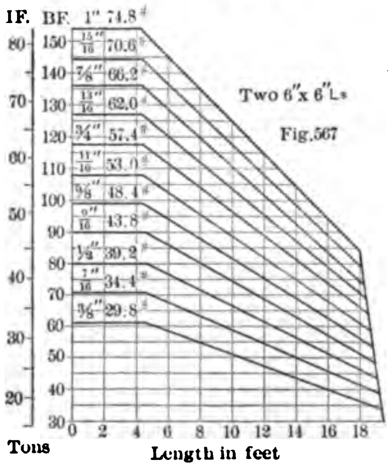


Fig.567

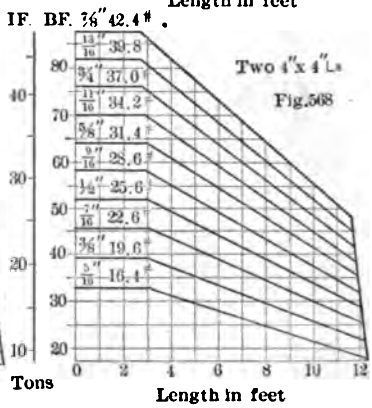


Fig.568

than pin connections for roof trusses. These tables are constructed and used in the same manner as the last two, but the safe length in feet is limited to 10 times the least radius of gyration $R°$ of cross-section, and which is perpendicular to webs. No deduction for rivets.

526. Example.—Strut 17 ft. long under compression of 35 tons. Inspection of the table shows that 2, 13″, 32 # channels, weight 64 # per foot are required, nearly three times as much weight as for a strut composed of two latticed channels. No smaller sizes of channels could safely be made 17 ft. long when riveted to gusset plates.

This surprising result shows clearly that it would be most economical to use latticed channels, or to design some mode of connecting them other than gusset plates.

527. Tables M, N, and O.—Two steel angles in compression riveted to $\frac{3}{4}''$ gusset plate set between them.

Table M is for angles with equal legs with one or both legs riveted, respectively marked *I F* and *B F* at upper ends of vertical load lines of the diagrams. For $8 \times 8''$ to $2 \times 2''$ angles.

Tables N and O are for angles with unequal legs, either with narrow leg, with wide leg, or with both legs riveted, respectively marked *N F*, *W F*, and *B F* at the upper ends of load lines of the diagrams.

The scales of the different load lines of each diagram are varied, so that a single diagram for each angle may be used for either mode of riveting at ends. No deduction is made for rivet holes.

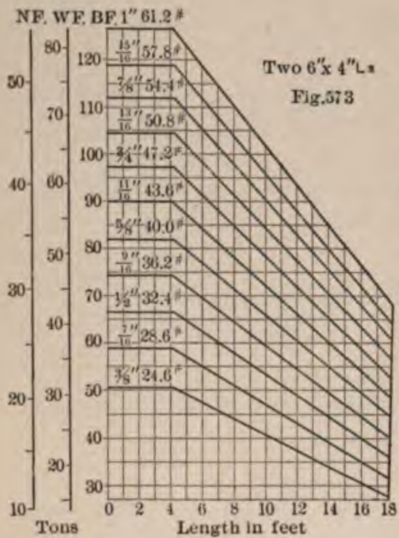
Length of member in feet is limited in the diagrams to 10 times the least radius $R°$ of its cross-section in inches, as usual.

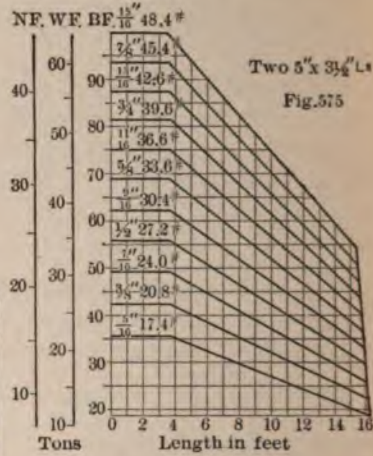
Maximum safe compression is here taken at 7 tons per sq. inch in accordance with the Chicago ordinance.

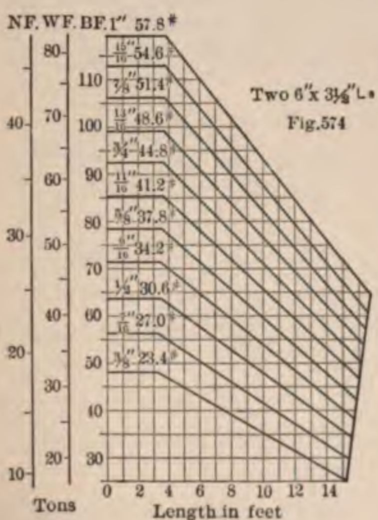
It is usually most economical to rivet both legs to the gusset plate, either directly or by means of a short piece of angle. Otherwise the wide leg should be directly riveted to it.

528. Example 1.—A strut 15 ft. long is subject to compression of 20 tons.

TABLE N—TWO ANGLES WITH GUSSET







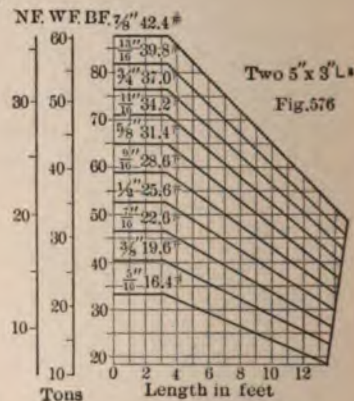
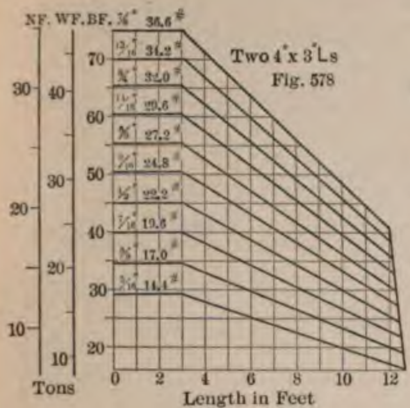
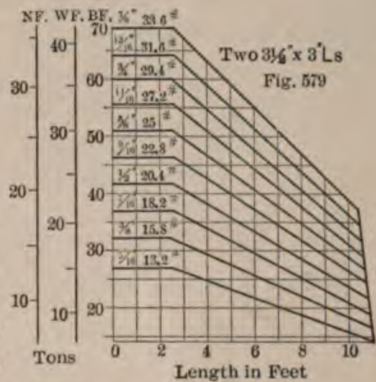
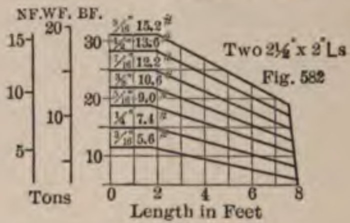
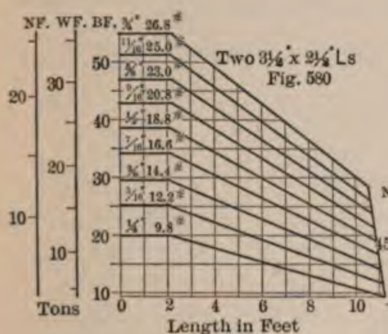


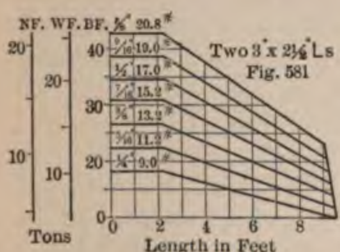
TABLE O—TWO ANGLES WITH GUSSET











By Table M, $2, 6 \times 6 \times \frac{3}{8}''$, 29.6 #, if both legs are riveted.

$2, 6 \times 6 \times \frac{7}{16}''$, 34.4 #, if one leg is riveted.

By table N, $2, 5 \times 3\frac{1}{2}''$, 24.0 #, if both legs are riveted.

$2, 5 \times 3\frac{1}{2} \times \frac{11}{16}''$, 36.6 #, if wide leg is riveted.

$2, 5 \times 3\frac{1}{2} \times \frac{15}{16}''$, 48.4 #, if narrow leg is riveted.

Table O contains no angles that could be made 15 ft. long safely. Hence it would be most economical to use $2, 5 \times 3\frac{1}{2} \times \frac{1}{2}''$ angles, 24.0 # per foot, with both legs riveted. The economy in riveting both legs of the angles is sufficiently evident.

529. Example 2.—A strut is composed of $2, 5 \times 3\frac{1}{2} \times \frac{7}{16}''$ angles and is required to safely resist a longitudinal compression of 20 tons. Required its maximum safe length.

By Table N, 15.4 ft. with both legs riveted.
14.3 ft. with wide leg riveted.
4.5 ft. with narrow leg riveted.

530. Table P.—Wooden posts, struts, or principals. For all woods in common use, with loads up to 60 or 100 tons and lengths not exceeding 35 feet.

Values of C in this table are taken from the Table of Coefficients for Woods in Chapter IX. Six columns at the left correspond to values for C of .50, .55, .60, .70, .75, and .80 ton compression endwise per sq. inch. The scale in each column is so varied that each can be used for the single diagram of sectional dimensions and lengths on the right. The safe lengths in feet are limited to 2½ times the least side in inches, as usual. Initials of woods below.

A horizontal through the load in tons, taken in the proper column for the kind of wood, intersects a vertical through the length in feet on or just below the inclined line representing the safe sectional dimensions. For woods with other coefficients, the dimensions can readily be found by interpolation.

531. Example 1.—A longleaf pine principal is required to safely resist a longitudinal compression of 80 tons. Its length between

TABLE P—SMALL WOODEN POSTS

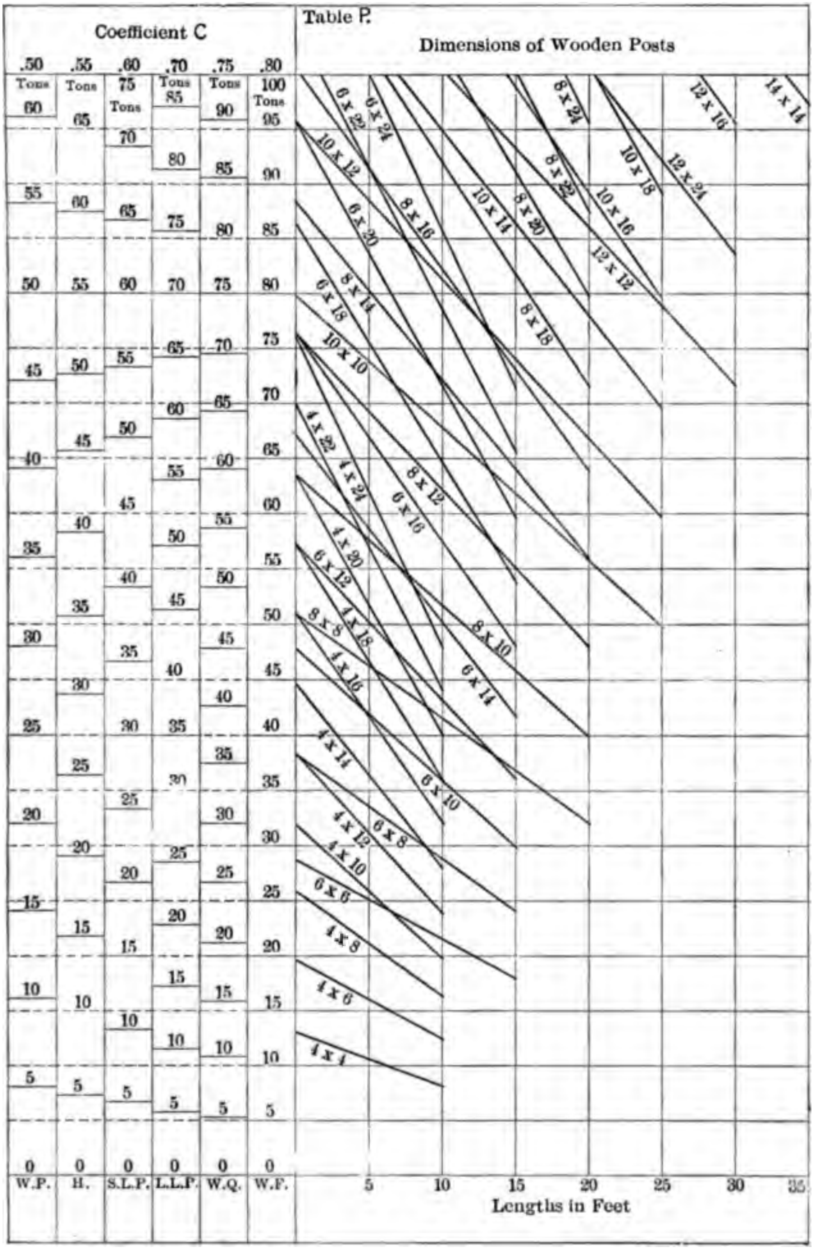
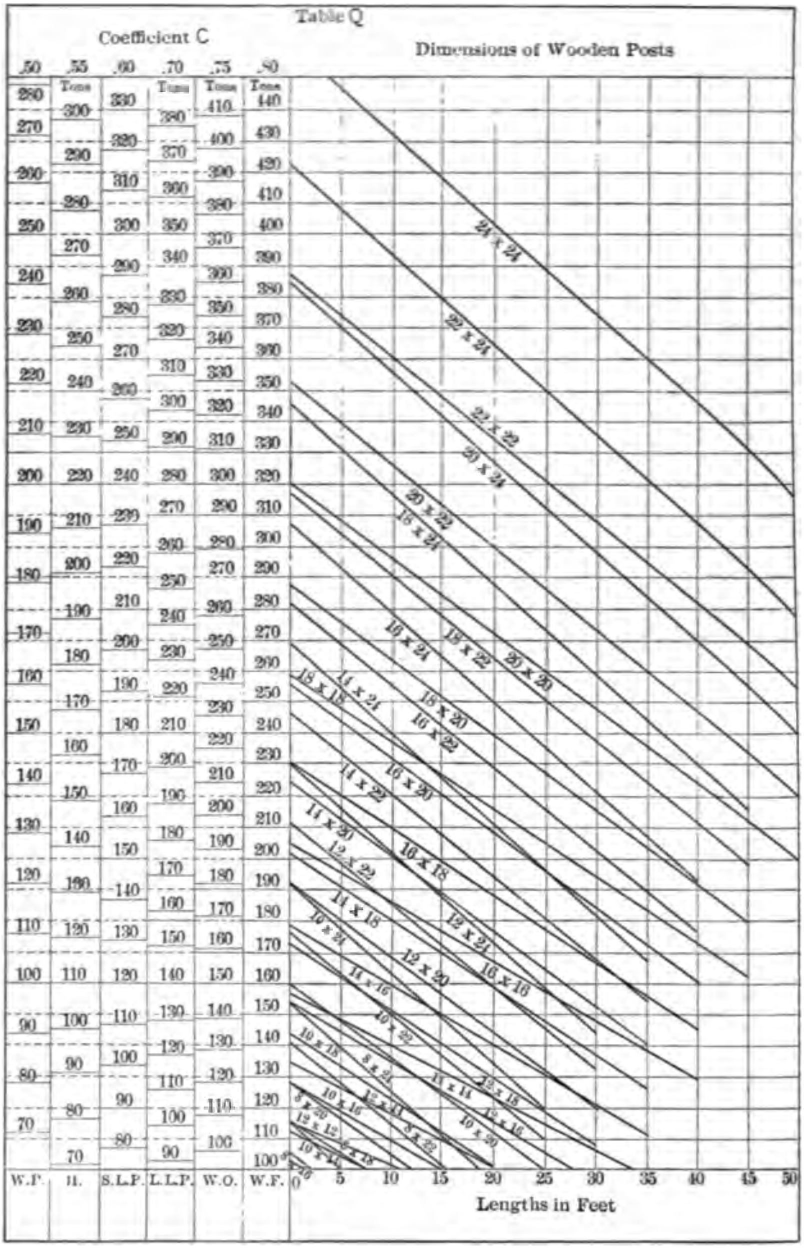


TABLE Q—LARGE WOODEN POSTS



apexes is 15 ft. C = .80 for longleaf pine. Taking 80 tons in column headed .80, a horizontal intersects a vertical through 15 ft. below the inclined lines for 8 × 18″, 10 × 14″, 8 × 20″, 12 × 12″, etc. Hence the size most conveniently obtained will be 10 × 14″, and it should be used.

532. Example 2.—A white-pine post is 18 ft. in clear length and is required to safely carry a load of 45 tons.

Value of C for white pine = .50, hence take column headed .50. By the table, a post 10 × 14″ or 12 × 12″ will suffice.

533. Table Q.—Wooden posts, struts, or principals. This is similar in construction to Table P, but is arranged for greater loads from 65 to 450 tons, and for lengths up to 50 feet. It is used in the same manner.

534. Example 1.—A shortleaf pine principal is 25 ft. in clear length and must safely resist a compression of 115 tons.

By Table Q, its dimensions must be 14 × 18″, 12 × 22″ or 16 × 16″. Either the first or last would be used.

535. Example 2.—A white-oak post is 20 ft. long and is 14 × 14″. What is its maximum safe load in tons?

According to Table Q it will carry 116 tons.

III. TRANSVERSE LOADS ON BEAMS, JOISTS, RAFTERS, ETC.

536. Sectional Dimensions.—A very convenient series of diagrams for such structural members of wood or steel shapes may be found in Winslow's Tables, which are also applicable to rafters and purlins of roofs. Diagrams for this purpose are not given here, especially since the simplified formulas given in Chapter IX are easily applied to any case and afford direct results.

By these formulas are to be computed the minimum safe numerical values of the section modulus and of the section moment of inertia required for the member. The actual sectional dimensions of the member are then easily obtained from Tables R and S for wood, and from Cambria and Carnegie for steel shapes.

537. Table R.—Section moduluses of wooden timbers. This table gives the value of section modulus $\frac{I}{c}$ for each regular size of timbers from 1 × 2″ to 24 × 24″.

TABLE R

MODULUS $\frac{I}{c}$ FOR RECTANGULAR SECTION

Depth of Section in Inches.	Breadth of Section in Inches.														
	1	1⅝	2	3	4	6	8	10	12	14	16	18	20	22	24
2	1	1	1	2	3	4	5	7	8	9	11	12	13	15	16
4	3	4	5	8	11	16	21	27	32	37	43	48	53	59	64
6	6	10	12	18	24	36	48	60	72	84	96	108	120	132	144
8	11	17	21	32	43	64	85	107	128	149	171	192	213	234	256
10	17	27	33	50	67	100	133	167	200	233	267	300	333	367	400
12	24	39	48	72	96	144	192	240	288	336	384	432	480	528	576
14	33	53	65	98	131	196	261	327	392	327	523	588	653	719	784
16	43	70	85	128	171	256	341	427	512	598	683	768	853	939	1024
18	54	88	108	162	216	324	432	540	648	756	864	972	1080	1188	1296
20	67	108	133	200	267	400	533	667	800	933	1067	1200	1333	1467	1600
22	81	131	161	242	323	484	645	807	968	1129	1291	1452	1614	1775	1936
24	96	156	192	288	384	576	768	960	1152	1344	1536	1728	1920	2112	2304

TABLE S

MOMENT OF INERTIA I FOR RECTANGULAR SECTION

Depth of Section in Inches.	Breadth of Section in Inches.														
	1	1⅝	2	3	4	6	8	10	12	14	16	18	20	22	24
2	1	1	1	2	3	4	5	7	8	9	11	12	13	15	16
4	5	9	11	16	21	32	43	53	64	75	85	96	107	117	128
6	18	29	36	54	72	108	144	180	216	252	288	324	360	396	432
8	43	79	85	128	171	256	341	427	512	597	683	768	853	939	1024
10	83	135	167	250	333	500	667	833	1000	1167	1333	1500	1667	1833	2000
12	144	234	288	432	576	864	1152	1440	1728	2016	2304	2592	2880	3168	3456
14	229	371	457	686	915	1372	1829	2287	2744	3201	3659	4116	4573	5031	5488
16	341	554	683	1024	1365	2048	2731	3413	4096	4779	5461	6144	6827	7509	8192
18	486	790	972	1458	1944	2916	3888	4860	5832	6804	7776	8748	9720	10692	11664
20	667	1083	1333	2000	2667	4000	5333	6667	8000	9333	10667	12000	13333	14667	16000
22	887	1441	1774	2662	3549	5324	7099	8873	10648	12423	14197	15972	17757	19521	21296
24	1152	1872	2304	3456	4608	6912	9216	11520	13824	16128	18432	20736	23040	25344	27648

538. Table S.—Section moments of inertia of wooden timbers. This table gives the numerical values of section moment of inertia I for each regular size of timbers from $1 \times 2''$ to $24 \times 24''$.

After obtaining the required numerical values of $\frac{I}{c}$ and I by the formulas in Chapter IX, the proper sectional dimensions of the member are found by these tables. The horizontal width of a timber is found at the top, the vertical depth being at the left, corresponding to the numerical values of $\frac{I}{c}$ or I required. The safest dimensions must be taken for use.

539. Example 1.—Assume that $\frac{I}{c}$ has been found to be 150 and $I = 900$.

For $\frac{I}{c}$: $3 \times 18''$, $4 \times 16''$, $6 \times 14''$, $8 \times 12''$, $10 \times 10''$, would suffice.

For I: $3 \times 16''$, $4 \times 14''$, $6 \times 14''$, $8 \times 12''$, $10 \times 12''$, would do.

Either $6 \times 14''$ or $8 \times 12''$ may be used, the first being more economical, because 84 sq. ins. in cross-section instead of 96 sq. ins. It is now difficult and more expensive to procure timbers 18″ or more in dimensions.

540. Example 2.—Assume a beam 10×12 ins. Required the corresponding values of $\frac{I}{c}$ and I.

By Table R, $\frac{I}{c} = 240$. By Table S, $I = 1440$.

For members composed of steel shapes, the required values of $\frac{I}{c}$ and I are given in Cambria and Carnegie.

Opposite the numerical value of $\frac{I}{c}$ in column S of Properties of Sections, find at the left the depth and weight of the shapes required to safely resist breaking; opposite numerical value of I in column I, find at left the depth and weight of shapes required for limited deflection.

541. Example 3.—Assume required values of $\frac{I}{c} = 35.0$; of $I = 215.0$.

For I-beams, $\frac{I}{c}$ requires 1, 10″ 30 # beam or 2, 9″ 21 # beams.

I requires 1, 12″ 31.5 #, or 2, 10″ 25 # beams.

TABLE T

RESISTANCE OF RIVETS TO SINGLE SHEAR AND BEARING—RIVETS THREE-FOURTHS OR SEVEN-EIGHTHS INCH IN DIAMETER.

No.	¾ in. Rivets			⅞ in. Rivets				No.	¾ in. Rivets			⅞ in. Rivets			
Rivets	3/16 in. Plate	¼ in. Plate	5/16 in. Plate and Over	3/16 in. Plate	¼ in. Plate	5/16 in. Plate	⅜ in. Plate and Over	Rivets	3/16 in. Plate	¼ in. Plate	5/16 in. Plate	3/16 in. Plate	¼ in. Plate	5/16 in. Plate	⅜ in. Plate and Over
1	1.69	2.26	2.65	1.97	2.63	3.29	3.60	26	43.87	58.50	68.92	51.20	68.27	85.33	93.82
2	3.37	4.50	5.30	3.94	5.26	6.56	7.21	27	45.55	60.76	71.57	53.17	70.90	88.62	97.43
3	5.06	6.76	7.96	5.90	7.87	9.85	10.82	28	47.24	63.00	74.22	55.14	73.51	91.90	101.04
4	6.74	9.00	10.61	7.87	10.50	13.13	14.44	29	48.92	65.26	76.87	57.11	76.14	95.18	104.64
5	8.44	11.26	13.26	9.85	13.13	16.42	18.05	30	50.62	67.50	79.52	59.08	78.77	98.46	108.25
6	10.13	13.50	15.90	11.82	16.76	19.69	21.65	31	52.31	69.76	82.18	61.04	81.40	101.75	111.86
7	11.81	15.76	18.55	13.79	18.38	22.98	25.26	32	53.99	72.00	84.83	63.01	84.02	105.00	116.46
8	13.50	18.00	21.20	15.76	21.00	26.26	28.87	33	55.68	74.26	87.48	64.98	86.64	103.31	119.08
9	15.18	19.66	23.86	17.72	23.63	29.54	32.47	34	57.36	76.50	90.13	66.95	89.27	111.59	122.69
10	16.87	22.50	26.51	19.69	26.26	32.82	36.08	35	59.05	78.76	92.78	68.93	91.90	114.88	126.30
11	18.56	24.76	29.16	21.66	28.88	36.12	39.70	36	60.74	81.00	95.42	70.90	94.52	118.15	129.90
12	20.24	27.00	31.81	23.63	31.51	39.38	43.30	37	62.42	83.26	98.08	72.86	97.15	121.44	133.51
13	21.94	29.26	34.46	25.60	34.13	42.67	46.91	38	64.12	85.50	100.73	74.83	99.77	124.72	137.12
14	23.62	31.50	37.12	27.56	36.76	45.95	50.52	39	65.80	87.76	103.38	76.80	102.40	128.00	140.72
15	25.31	33.76	39.77	29.54	39.37	49.30	54.13	40	67.49	90.00	106.03	78.77	105.02	131.28	144.34
16	27.00	36.00	42.41	31.63	42.01	52.51	57.73	41	69.18	92.26	108.68	80.74	107.65	134.51	147.95
17	28.68	38.26	45.06	33.48	44.64	55.80	61.34	42	70.86	94.50	111.34	82.70	110.28	137.84	151.55
18	30.37	40.50	47.71	35.45	47.26	59.08	64.96	43	72.55	96.76	113.99	84.67	112.90	141.13	155.16
19	32.50	42.76	50.36	37.42	49.88	62.33	68.54	44	74.23	99.00	116.64	86.64	115.52	144.41	158.77
20	33.74	45.00	53.02	39.38	52.51	65.64	72.17	45	75.92	101.26	119.29	88.62	118.15	147.70	162.38
21	35.44	47.26	55.67	41.35	55.14	68.93	75.78	46	77.62	103.50	121.93	90.59	120.78	150.97	165.98
22	37.12	49.50	58.32	43.32	57.77	72.20	79.38	47	79.30	105.76	124.58	92.56	123.41	154.26	169.60
23	38.81	51.76	60.97	45.29	60.38	75.49	82.99	48	80.99	108.00	127.24	94.52	126.02	157.54	173.21
24	40.49	54.00	63.62	47.26	63.01	78.18	86.60	49	82.67	110.26	129.88	96.49	128.65	160.82	176.81
25	42.18	56.26	66.28	49.24	65.64	82.06	90.22	50	84.36	112.50	132.54	98.46	131.28	164.10	180.42

Since a single I-beam is in danger of collapsing sidewise unless stayed at several points in its length by other members, it is often necessary to use 2 I-beams connected by bolts and separators, although less economical in weight and cost.

For channels, $\frac{I}{c}$ requires 1, 15″ 33 #, or 2, 12″ 20.5 #.

I requires 1, 15″ 33 #, or 2, 12″ 20.5 #.

In this case 2 channels would be preferable to 2 I-beams, since 9 # per lineal foot would be saved.

No smaller shapes have sufficient values of $\frac{I}{c}$ and I.

542. Table T.—Rivet table for $\frac{3}{4}$″ and $\frac{7}{8}$″ rivets. This table is based on the safe resistance of rivets to single shear of 6 tons per square inch, and their safe resistance to bearing of 12.5 tons per square inch, according to Chicago ordinance.

The table gives the number of rivets required to transmit the given stress in tons, by single shear, when the plates connected are sufficiently thick to develop the full resistance of the rivet to shear, as well as the bearing resistance of thinner plates. Diameters of rivets and thickness of plates are found at top, number of rivets required at the left, the maximum safe resistances in tons being given in the body of the table.

543. Example 1.—A stress of 50 tons is to be transmitted from a member composed of two angles to a gusset plate inserted between them. $\frac{3}{4}$″ rivets. Gusset $\frac{5}{8}$″ thick.

There is a single shear of $\frac{50}{2} = 25$ tons on each side of gusset.

By Table T, 10, $\frac{3}{4}$″ rivets, if legs are $\frac{5}{16}$″ thick or more.

12, $\frac{3}{4}$″ rivets, if legs are $\frac{1}{4}$″ inch thick.

15, $\frac{3}{4}$″ rivets, if legs are $\frac{3}{16}$″ thick.

The procedure is similar for $\frac{7}{8}$″ rivets, which are usually employed only for very heavy structures.

TABLE U

STEEL PINS—SAFE RESISTANCES TO SHEAR, BEARING, BENDING MOMENTS

Diam. Rough	Diam. Finished	Shear Tons	Bearing Tons per Inch Long	Bending Mom. Inch Tons	Diam. Rough	Diam. Finished	Shear Tons	Bearing Tons per Inch Long	Bending Mom. Inch Tons	Diam. Rough	Diam. Finished	Shear Tons	Bearing Tons per Inch Long	Bending Mom. Inch Tons
1	15/16	4.14	11.72	1.01	3 3/4	3 11/16	64.08	46.09	61.53	6 1/2	6 7/16	195.29	80.47	327.39
1 1/8	1 1/16	5.32	13.28	1.47	3 7/8	3 13/16	68.50	47.66	68.01	6 5/8	6 9/16	202.94	82.03	346.84
1 1/4	1 3/16	6.65	14.84	2.06	4	3 15/16	73.06	49.22	74.92	6 3/4	6 11/16	210.75	83.59	367.04
1 3/8	1 5/16	8.12	16.41	2.78	4 1/8	4 1/16	77.77	50.78	82.28	6 7/8	6 13/16	218.71	85.16	388.00
1 1/2	1 7/16	9.74	17.97	3.65	4 1/4	4 3/16	82.63	52.34	90.11	7	6 15/16	226.80	86.72	409.76
1 5/8	1 9/16	11.51	19.53	4.68	4 3/8	4 5/16	87.64	53.91	98.43	7 1/8	7 1/16	235.05	88.28	432.31
1 3/4	1 11/16	13.42	21.09	5.46	4 1/2	4 7/16	92.79	55.47	107.23	7 1/4	7 3/16	243.54	89.84	455.72
1 7/8	1 13/16	15.48	22.66	7.31	4 5/8	4 9/16	98.10	57.03	116.55	7 3/8	7 5/16	251.98	91.41	479.86
2	1 15/16	17.69	24.22	8.93	4 3/4	4 11/16	103.54	58.59	126.40	7 1/2	7 7/16	260.68	92.97	504.89
2 1/8	2 1/16	20.05	25.78	10.77	4 7/8	4 13/16	109.14	60.16	136.78	7 5/8	7 9/16	269.48	94.53	530.78
2 1/4	2 3/16	22.55	27.34	12.85	5	4 15/16	114.88	61.72	147.72	7 3/4	7 11/16	278.49	96.09	557.53
2 3/8	2 5/16	25.20	28.91	15.18	5 1/8	5 1/16	120.77	63.28	159.22	7 7/8	7 13/16	287.62	97.66	585.18
2 1/2	2 7/16	28.00	30.47	17.77	5 1/4	5 3/16	126.81	64.84	171.31	8	7 15/16	296.90	99.22	613.71
2 5/8	2 9/16	30.94	32.03	20.65	5 3/8	5 5/16	133.00	66.41	184.00	8 1/8	8 1/16	306.32	100.78	643.16
2 3/4	2 11/16	34.04	33.59	23.82	5 1/2	5 7/16	139.33	67.97	197.29	8 1/4	8 3/16	315.89	102.34	673.54
2 7/8	2 13/16	37.28	35.16	27.30	5 5/8	5 9/16	145.81	69.53	211.22	8 3/8	8 5/16	325.61	103.91	704.86
3	2 15/16	40.66	36.72	31.11	5 3/4	5 11/16	152.44	71.09	225.78	8 1/2	8 7/16	335.48	105.47	737.15
3 1/8	3 1/16	44.20	38.28	35.26	5 7/8	5 13/16	159.21	72.66	240.99	8 5/8	8 9/16	345.50	107.03	770.40
3 1/4	3 3/16	47.88	39.84	39.74	6	5 15/16	166.13	74.22	256.88	8 3/4	8 11/16	355.66	108.59	804.63
3 3/8	3 5/16	51.71	41.41	44.61	6 1/8	6 1/16	173.20	75.78	273.45	8 7/8	8 13/16	365.96	110.16	839.86
3 1/2	3 7/16	55.68	42.97	49.85	6 1/4	6 3/16	180.41	77.34	290.71	9	8 15/16	376.42	111.72	876.11
3 5/8	3 9/16	59.81	44.53	55.49	6 3/8	6 5/16	187.78	78.91	308.69					

544. Example 2. Inspection of a Structure.—5, $3\frac{3}{4}''$ rivets connect a member composed of 2, $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}''$ angles to a $\frac{5}{8}''$ gusset plate. Required the maximum total safe stress that may be transmitted.

By the table, 11.26 tons in single shear or 22.52 tons in double shear. If the stress in the member were greater, the stress in the rivets would exceed its safe limits.

545. Table U.—For steel pins for connecting truss members at apexes. This table is based on a safe resistance to single shear of 6 tons per square inch, of safe resistance to bearing of 12.5 tons per square inch, and a maximum safe fibre stress of 12.5 tons per square inch, in accordance with Chicago ordinance.

The table gives for steel pins from 1 to 6 ins. diameter their safe resistance to transverse single shear, to bearing per inch length of pin, and their maximum safe bending moments in inch-tons. After determining the single shear, bearing stress, and bending moment, the required diameter of pin is easily found by inspection. Finished sizes of pins are usually $\frac{1}{16}$ inch less than rough sizes.

546. Example. Pin Connection at Apex of Truss.—Assume single shear of 12.5 tons; bearing of 25 tons on one inch length of pin; bending moment of 15 inch-tons.

By Table U, $1\frac{3}{4}''$ pin to resist shearing.

$2\frac{1}{8}''$ pin to resist bearing.

$2\frac{3}{8}''$ pin to resist bending moment.

Therefore a pin $2\frac{3}{8}''$ diameter is to be used.

IV. WEIGHTS OF STEEL RODS, ETC.

547. Purpose of the Tables.—In estimating the weight of a wooden truss with steel rods resisting tension, it is necessary to carefully compute the weight of each rod, its upset ends, nuts and cast-iron washers. The weight of the latter varies inversely as the

TABLE V

WEIGHTS OF ROD, ROD ENDS, NUTS AND WASHERS

Diameter	Weights of			Weights of Two Washers						
Rod	Lbs. per ft.	Two Ends	Two Nuts	C= .30	C= .25	C= .20	C= .175	C= .15	C= .125	C= .10
1/2"	0.67	1.06	0.63	1.66	1.93	2.35	2.59	2.95	3.45	4.19
9/16	0.85	1.06	0.63	2.00	2.34	2.81	3.17	3.62	4.23	5.17
5/8	1.04	1.53	0.93	2.83	3.28	3.90	4.40	5.64	5.99	7.32
11/16	1.26	1.99	1.42	4.02	4.66	5.61	6.29	7.17	8.42	10.24
3/4	1.50	1.99	1.42	4.62	5.35	6.53	7.29	8.19	9.81	12.17
13/16	1.76	2.67	2.04	6.33	7.35	8.85	9.98	11.38	12.82	16.40
7/8	2.04	3.30	3.13	8.33	9.78	10.89	13.02	14.85	17.29	21.23
15/16	2.35	3.30	3.13	9.31	10.94	13.05	14.68	16.73	19.68	23.98
1	2.67	4.21	5.00	12.39	14.77	17.85	19.08	21.75	25.48	30.89
1 1/16	3.02	4.21	5.00	13.63	15.73	19.13	21.18	24.12	28.24	34.38
1 1/8	3.38	5.01	6.35	16.88	19.40	23.18	26.00	29.68	34.54	42.20
1 3/16	3.77	5.01	6.35	18.02	21.17	25.48	28.56	32.60	38.16	47.19
1 1/4	4.17	6.16	9.09	22.12	25.63	30.79	34.56	39.32	45.91	55.91
1 5/16	4.60	7.15	11.11	22.67	30.81	36.98	40.32	47.10	55.20	66.65
1 3/8	5.05	7.15	11.11	28.72	33.44	39.94	44.67	50.94	60.33	72.31
1 7/16	5.52	8.60	13.33	34.94	39.61	47.64	52.04	60.66	71.04	86.33
1 1/2	6.01	9.79	16.33	39.84	46.05	55.01	62.63	70.40	82.35	100.55
1 9/16	6.52	9.79	16.33	42.48	49.17	58.60	66.20	75.50	88.47	108.06
1 5/8	7.05	11.56	17.35	47.95	55.61	66.72	75.16	85.73	100.68	122.46
1 11/16	7.60	11.56	17.35	50.69	59.22	71.49	80.16	91.64	107.58	131.30
1 3/4	8.18	12.96	19.94	58.49	67.89	79.71	91.69	104.65	122.80	149.81
1 13/16	8.77	12.96	19.94	61.90	71.79	86.87	97.42	111.32	130.17	159.76
1 7/8	9.39	15.06	20.56	68.39	80.03	94.20	108.65	124.17	145.99	178.91
1 15/16	10.02	16.69	24.47	77.97	91.22	106.93	123.02	140.62	164.63	201.35
2	10.68	16.69	24.47	81.69	95.42	117.76	129.37	147.71	175.12	213.27
2 1/8	12.06	19.16	24.82	97.05	109.87	135.16	152.11	173.54	204.74	252.34
2 1/4	13.52	23.92	34.21	115.42	134.51	157.56	183.34	210.16	241.67	309.42
2 3/8	15.06	26.05	39.31	138.26	161.22	195.13	217.73	250.54	295.17	361.26
2 1/2	16.69	29.35	44.74	158.89	185.34	224.87	252.24	288.72	339.76	420.54
2 5/8	18.40	31.75	50.89	177.91	213.66	258.42	293.57	332.53	396.07	481.53
2 3/4	20.19	35.50	57.29	207.70	241.91	294.25	359.56	373.67	445.49	546.63
2 7/8	22.07	42.40	66.79	269.41	275.29	346.85	386.69	440.75	541.79	674.80
3	24.03	46.95	74.65	302.56	318.81	386.36	434.84	500.05	584.94	718.26

value of C', the coefficient of the safe resistance of the wood to compression across the fibres. To avoid this labor in computation, Tables V and W were devised by the author.

TABLE W

WEIGHTS OF ROD, NUTS AND WASHERS

Diameter	Weights of		Weights of Two Washers						
Rod	Lbs. per ft.	Two Nuts	C= .30	C= .25	C= .20	C= .175	C= .15	C= .125	C= .10
½	0.67	0.17	0.66	0.76	0.92	1.05	1.19	1.41	1.72
$\frac{9}{16}$	0.85	0.33	1.02	1.17	1.41	1.59	1.81	2.11	2.48
⅝	1.04	0.35	1.32	1.55	1.87	2.09	2.41	2.87	3.46
$\frac{11}{16}$	1.26	0.50	1.77	2.06	2.49	2.80	3.21	3.78	4.62
¾	1.50	0.65	2.36	2.76	3.48	3.73	4.28	5.03	6.17
$\frac{13}{16}$	1.76	0.71	2.85	3.35	4.06	4.58	5.27	6.14	7.60
⅞	2.04	1.06	3.81	4.43	5.35	6.03	6.90	8.13	9.95
$\frac{15}{16}$	2.35	1.14	4.61	5.38	6.51	7.33	8.41	9.91	12.05
1	2.67	1.36	5.70	6.66	8.07	9.07	10.42	12.67	16.06
$1\frac{1}{16}$	3.02	1.45	6.82	7.97	9.69	10.92	12.50	14.79	18.07
1⅛	3.38	2.06	8.06	9.38	11.25	12.84	14.70	17.27	21.19
$1\frac{3}{16}$	3.77	2.18	9.44	11.04	13.41	15.13	17.38	20.41	25.18
1¼	4.17	2.90	11.49	13.33	16.12	18.20	20.84	24.53	30.08
$1\frac{5}{16}$	4.60	3.90	13.80	16.02	19.22	21.76	24.94	29.32	35.88
1⅜	5.05	4.94	15.60	18.18	21.88	24.58	28.05	32.90	40.16
$1\frac{7}{16}$	5.52	5.16	17.74	20.61	24.93	28.05	32.03	37.68	46.10
1½	6.01	6.38	20.71	24.07	29.83	32.66	37.29	43.81	53.50
$1\frac{9}{16}$	6.52	6.65	23.74	27.09	32.75	36.99	42.09	44.30	60.57
1⅝	7.05	8.00	26.40	30.46	36.81	41.27	47.18	55.44	67.68
$1\frac{11}{16}$	7.60	8.31	31.47	34.22	41.29	46.46	53.05	62.41	76.33
1¾	8.18	10.00	32.63	37.90	45.74	51.33	59.02	68.48	84.05
$1\frac{13}{16}$	8.77	10.35	36.07	41.93	50.78	57.86	65.20	76.66	93.28
1⅞	9.39	12.40	41.12	47.72	59.67	64.75	73.94	86.91	106.05
$1\frac{15}{16}$	10.02	12.80	45.07	52.48	63.47	71.26	81.52	95.84	117.19
2	10.68	15.00	49.13	60.35	68.91	77.40	89.59	103.91	126.90
2⅛	12.06	15.60	58.26	68.26	82.97	92.82	106.51	125.26	153.20
2¼	13.52	18.60	75.48	82.21	89.50	110.59	128.12	150.98	184.55
2⅜	15.06	19.20	79.64	93.00	112.92	126.35	145.57	171.56	210.21
2½	16.69	22.60	94.85	110.50	133.57	150.92	173.05	203.75	249.36
2⅝	18.40	24.50	109.25	127.81	155.21	174.95	200.99	237.65	290.72
2¾	20.19	27.40	126.99	147.93	180.79	206.15	233.65	275.63	338.18
2⅞	22.07	31.50	147.17	172.66	204.74	235.94	270.47	320.57	391.66
3	24.03	32.20	161.07	189.87	229.55	258.82	297.47	350.74	432.93

548. Table V.—Weight of rods with upset ends, etc. This table gives in successive columns the weight of 1 ft. of rod, of 2 upset ends, of 2 nuts, and of 2 cast-iron washers for

values of C', varying from 0.30 to 0.10, for rods from $\frac{1}{2}$ to 3 inches diameter.

549. Table W.—Weight of rods without upset ends, etc. This table omits the column for weight of 2 upset ends, and contains the other columns of Table V, but the corresponding values in the table differ from the weights given in Table V, since the safe tensile strength of the rods of same diameter is less, nuts and washers are smaller, etc.

It is unnecessary to give examples of the application of these tables, since their use is self-evident. It will be illustrated in Chapter XIV on Computing Weights of Trusses.

CHAPTER XI

DIMENSIONING MEMBERS OF ROOFS AND TRUSSES

550. General Remarks.—The dimensions of rafters and purlins are obtained by the formulas of Chapter IX; those of truss members by inspection of the tables of Chapter X.

551. Probable Maximum Stresses.—In American practice, it is common to take the sum of the permanent, snow, and wind stresses occurring in a member as the maximum stress in that member, dimensioning the member in accordance with this maximum. This method was pursued in the stress sheets of the examples treated in Chapter IV. But the author believes it scarcely possible for the maximum snow load and maximum wind load to act together on a roof at one time. Therefore it appears sufficient to dimension members of the roof and of the truss for the maximum of the permanent and snow or of the permanent and wind stresses, always taking the larger sum. Hence this system is followed in this chapter. Stresses produced by ceiling, crane, and other extra loads must be added to this maximum. But if any one prefers to regard all loads as being supported at one time by the roof, the maximum stresses may be taken from Chapter IV, and the members be dimensioned in the manner explained in this chapter. Most of the examples considered here are taken from Chapter IV.

552. Example 1. A Wooden Truss with Extra Loads.—Resume Example 1 of Chapter IV, a wooden truss supporting P, S, W, ceiling, and crane loads, constructed of shortleaf pine, excepting vertical steel rods.

553. Sheathing.—Usually $\frac{7}{8}''$ thick.

2 # (tin and paint) + 3 # (sheathing = 5 #) = P load per sq. ft. of roof.

Cos 21.8° × 17.53 = 16.28 # = S load per sq. ft. of roof.

14.53 # = wind load per sq. ft. of roof.

Let w' = component of maximum load acting perpendicular to sheathing.

w'' = component of maximum load acting parallel to sheathing.

Since the parallel component is resisted by the edgewise stiffness of the sheathing, it may evidently be neglected here.

Then $w' = \cos 21.8° \ (5 \# + 16.28 \#) = 19.76 \#$, which exceeds $\cos 21.8° \times 5 \# + 14.53 = 19.17 \#$, and is therefore to be taken as the maximum normal or perpendicular component per sq. ft. of roof.

Apply formulas 107 and 110 of Chapter IX.

$L = 51.6\, t \sqrt{\frac{F}{w'}} = 51.6 \times .875 \sqrt{\frac{.55}{19.76}} = 7.56$ ft. between centres of rafters.

$L = 1.44\, t \sqrt[3]{\frac{E}{w'}} = 1.44 \times .875 \sqrt[3]{\frac{600}{19.76}} = 3.93$ ft. between centres of rafters.

Hence the safe resistance of sheathing to bending requires that the rafters be not set over 3.93 ft. on centres.

554. Rafters.—

$2 \# + 3 \# + 3 \#$ (rafters) $= 8 \#$ = permanent load per sq. ft. of roof.

$w' = \cos 21.8° \ (8 \# + 16.28 \#) = 22.54 \#$ = maximum normal component of load.

$w'' = \sin 21.8° \ (8 \# + 16.28 \#) = 9.02 \#$ = maximum parallel component of load.

a. Assume rafters to be 2×8, full size.

Apply formulas 91 and 95 of Chapter IX.

$$e = \frac{16000\, F I}{w' L^2 c} = \frac{16000 \times .55 \times 21}{22.54 \times 10.77^2} = 70.7 \text{ ins. between centres of rafters.}$$

$$e = \frac{35.56\, E I}{w L^3} = \frac{35.56 \times 600 \times 85}{22.54 \times 10.77^3} = 64.5 \text{ ins. between centres of rafters.}$$

The value of $\frac{I}{c}$ is taken from Table R; of I from Table S.

b. Assume rafters to be $1\frac{5}{8} \times 7\frac{1}{2}''$ (common dimensions in Western States instead of 2×8), the ordinary size.

$$e = \frac{16000\, F I}{w' L^2 c} = \frac{16000 \times .55 \times 17}{22.54 \times 10.77^2} = 57.2 \text{ ins. on centres.}$$

$$e = \frac{35.56\, E I}{w' L^3} = \frac{35.56 \times 600 \times 79}{22.54 \times 10.77^3} = 60.0 \text{ ins. on centres.}$$

c. Assume rafters to be 2 × 6, full size.

$$e = \frac{16000\,F\,I}{w'\,L^2\,c} = \frac{16000 \times .55 \times 12}{22.54 \times 10.77^2} = 40.4 \text{ ins. on centres.}$$

$$e = \frac{35.56\,E\,I}{w'\,L^3} = \frac{35.56 \times 600 \times 36}{22.54 \times 10.77^3} = 27.3 \text{ ins. on centres.}$$

d. Assume rafters to be $1\frac{5}{8} \times 5\frac{3''}{4}$, ordinary size.

$$e = \frac{16000\,F\,I}{w'\,L^2\,c} = \frac{16000 \times .55 \times 10}{22.54 \times 10.77^2} = 33.7 \text{ ins. on centres.}$$

$$e = \frac{35.56\,E\,I}{w'\,L^3} = \frac{35.56 \times 600 \times 29}{22.54 \times 10.77^3} = 22.0 \text{ ins. on centres.}$$

Hence it is best to use 2 × 6 rafters, spaced 27″ or 22″ on centres, according to whether they are full or ordinary size. They are here assumed to be full size and set 27″ on centres.

The parallel component of the load = 9.02 # and produces longitudinal compression in the rafter.

Then $\frac{w'\,L\,e}{2 \times 2000} = \frac{9.02 \times 10.77 \times 2.25'}{2 \times 2000} = 0.0547$ ton = longitudinal compression at mid-length section of rafter.

And $\frac{0.0547}{2 \times 6} = 0.00456$ ton = uniform compression per sq. inch.

Compute maximum deflection of rafter by the following formula:

$$\Delta = \frac{w'\,L^4\,e}{1067\,E\,I} = \frac{22.54 \times 10.77^4 \times 27}{10.67 \times 600 \times 36} = 0.355 \text{ inch at middle.}$$

Apply formula 141 to determine maximum fibre stress at section.

$$0.00456\left(1 + \frac{6 \times 0.355}{6}\right) = 0.00618 \text{ ton per sq. inch.}$$

Then $F' = F - 0.00618 = 0.55 - 0.00618 = 0.54382.$

Apply formula 89 with value of F' instead of F.

$$\frac{I}{c} = \frac{w'\,L^2\,e}{16000\,F'} = \frac{22.54 \times 10.77^2 \times 27}{16000 \times 0.54382} = 8.12.$$

But the actual value of $\frac{I}{c}$ is 12 for the given section; hence the rafter is entirely safe for both components w' and w''. The parallel component w'' of the maximum load on rafters may generally be neglected, excepting for unusually steep roofs.

555. Purlins.—
2# + 3# + 3# + 3# (purlin) = 11# = permanent load per sq. ft. of roof.

$w' = \cos 21.8° (11\# + 16.28\#) = 25.33\#$ = maximum normal component per sq. ft.

$w'' = \sin 21.8° (11\# + 16.28\#) = 10.13\#$ = maximum parallel component per sq. ft.

556. Purlin Set with Sides Perpendicular to Roof Surface.—
$W' = A\, w' = 161.55 \times 25.33 = 4092\# = 2.046$ tons = normal component of load on purlin.

$W'' = A\, w'' = 161.55 \times 10.13 = 1636\# = 0.818$ ton = parallel component of load on purlin.

Apply formulas 77 and 80 to both components.

For normal component W':

$$\frac{I}{c} = \frac{3\, W' L}{2\, F} = \frac{3 \times 2.046 \times 15}{2 \times .55} = 83.7.$$

$$I = \frac{675\, W'\, L^2}{E} = \frac{675 \times 2.046 \times 15^2}{600} = 518.0.$$

For parallel component W'':

$$\frac{I}{c} = \frac{3\, W'' L}{2\, F} = \frac{3 \times 0.818 \times 15}{2 \times .55} = 33.5.$$

$$I = \frac{675\, W'' L^2}{E} = \frac{675 \times 0.818 \times 15^2}{600} = 207.0.$$

For normal component W':

By Table R: 4 × 12″, 6 × 10″, 10 × 8″, would suffice.

By Table S: 4 × 12″, 6 × 12″, 8 × 10″, 10 × 10″.

For parallel component W''.

By Table R: 14 × 4″, 12 × 6″, 10 × 6″, 8 × 6″.

By Table S: 14 × 6″, 12 × 6″, 10 × 8″, 8 × 8″.

Evidently 6 × 12″ would be sufficient for each of the four cases, if the resultant W of the components W' and W'' coincided with the major axis of the section of the purlin. But this is not the condition here, and a correction must be applied. Fig. 585 represents this method.

Apply the method of correction given in Art. 476 of Chapter IX.

In Fig. 585, $K\ L\ M\ N$ is the cross-section of the purlin, its major axis being $X\ X$ and minor axis $Y\ Y$.

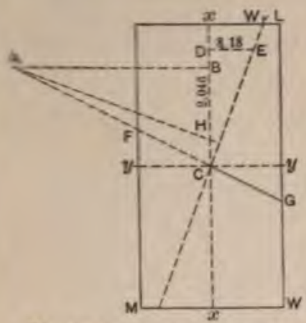


FIG. 585.—Section of Purlin

Laying off $C\ D = W' = 2.046$ tons, and $D\ E = W'' = 0.818$ ton, $E\ C$ is the line of action of their resultant W. By Table S, $I_y = 864$ and $I_x = 216$ for a section $8 \times 12''$. Make $C\ B = 864$ and $C\ H = 216$ at any convenient scale. Draw $B\ A$ horizontal and $H\ A$ perpendicular to $E\ C$. Join $A\ C$, and $F\ G$ is the neutral axis of the section under these conditions. Evidently the fibre stress at L and M is greater than if the resultant W coincided with the axis $X\ X$ and neutral axis $F\ G$ with $Y\ Y$.

Apply formula 145 to determine this maximum fibre stress.

$$.75\,L\left(\frac{W'\,d}{I_y}+\frac{W''\,b}{I_x}\right)=.75\times 15\left(\frac{2.046\times 12}{864}+\frac{0.818\times 6}{216}\right)=0.576,\text{which}$$

exceeds the safe value of $F = 0.55$ ton per square inch.

Hence these purlins must be made $8 \times 12''$. $I_y = 1152$ and $I_x = 512$.

$$.75\,L\left(\frac{W'\,d}{I_y}+\frac{W''\,b}{I_x}\right)=.75\times 15\left(\frac{2.046\times 12}{1152}+\frac{0.818\times 8}{512}\right)=0.384.$$

Therefore a purlin $8 \times 12''$ deep is entirely safe.

557. Purlin with Sides Vertical. — For permanent and snow loads:

$w' = 11\,\# + 16.28\,\# = 27.28\,\#$ per sq. ft. of roof surface.
$w'' = 0.$
$W' = A\,w' = 161.55 \times 27.28\,\# = 4407\,\# = 2.204$ tons.
$W'' = A\,w'' = 161.55 \times 0 = 0.$

For permanent and wind loads:

$w' = 11\,\# + \cos 21.8° \times 14.53\,\# = 24.49\,\#$ per sq. ft. of roof surface.
$w'' = \sin 21.8° \times 14.53\,\# = 5.40\,\#$ per sq. ft. of roof surface.
$W' = A\,w' = 161.55 \times 24.49\,\# = 3956\,\# = 1.978$ tons.
$W'' = A\,w'' = 161.55 \times 5.40\,\# = 872\,\# = 0.436$ ton.

The purlin must safely resist both pairs of components W' and W'' acting at different times.

For permanent and snow loads:

$$\frac{I}{c} = \frac{3\,W'L}{2\,F} = \frac{3 \times 2.204 \times 15}{2 \times .55} = 90.2.$$

$$I = \frac{675\,W'L^2}{E} = \frac{675 \times 2.204 \times 15^2}{600} = 558.$$

By Table R: $4 \times 12''$, $6 \times 10''$, $8 \times 10''$, $10 \times 8''$.

By Table S: $4 \times 12''$, $6 \times 12''$, $8 \times 10''$, $10 \times 10''$.

For permanent and wind loads:

$$\frac{I}{c} = \frac{3\,W'L}{2\,F} = \frac{3 \times 1.978 \times 15}{2 \times .55} = 80.8.$$

$$I = \frac{675\,W'L^2}{E} = \frac{675 \times 1.978 \times 15^2}{600} = 500.$$

$$\frac{I}{c} = \frac{3\,W''L}{2\,F} = \frac{3 \times 0.436 \times 15}{2 \times .55} = 17.8.$$

$$I = \frac{675\,W''L^2}{E} = \frac{675 \times 0.436 \times 15^2}{600} = 111.$$

By Table R: $4 \times 12''$, $6 \times 10''$, $8 \times 8''$, $10 \times 8''$.
By Table S: $4 \times 12''$, $6 \times 10''$, $8 \times 10''$, $10 \times 10''$.
By Table R: $8 \times 4''$, $10 \times 4''$, $12 \times 4''$.
By Table S: $8 \times 6''$, $10 \times 6''$, $12 \times 6''$.

Evidently a purlin $6 \times 12''$ will suffice for either case considered, unless the fibre stresses for permanent and wind loads exceed the safe limit of 0.55 ton per square inch.

$$.75\,L\left(\frac{W'd}{I_y} + \frac{W''b}{I_x}\right) = .75 \times 15\left(\frac{1.978 \times 12}{864} + \frac{0.436 \times 6}{216}\right) = 0.4453.$$

Hence this size of purlin is amply safe.

558. Ceiling Joists.—These must be set 16″ on centres to receive laths and plaster.

$w = 10$ # (laths and plaster) $+2$ # (joists) $+3$ # (floor) $= 15$ # per sq. ft.

Apply formulas 89 and 93.

$$\frac{I}{c} = \frac{w\,L^2\,e}{16000\,F} = \frac{15 \times 15^2 \times 16}{16000 \times 0.55} = 6.14.$$

$$I = \frac{w\,L^3\,e}{35.56\,E} = \frac{15 \times 15^3 \times 16}{35.56 \times 600} = 38.0.$$

By Table R: $2 \times 6''$, or even $1\frac{5}{8} \times 5\frac{3}{4}''$.

By Table S: $2 \times 8''$, or even $1\frac{5}{8} \times 7\frac{1}{2}''$.

Hence these ceiling joists should be made 2 × 8″, full or ordinary size. This provides for no live load on the floor, which is merely to protect the plastering.

559. Truss Members.—Resume the stress sheet of Example 1 of Chapter IV and determine the maximum stress in each member, assuming that snow and wind loads are not supported at the same time.

560. Upper Chord.—Apply Table P, Fig. 583. Use column headed .60, since this is the value of C for shortleaf pine.

Member X 1: centre length = $10'\ 9\frac{1}{4}''$; maximum stress = − 55.9 tons; 10 × 12″. This may be set with the 10″ dimension vertical, in order to make the chords 12″ wide horizontally, so that the lower chord need not be so deep, or the upper chord may best be made 12 × 12″, to allow for boxing ends of web struts, holes for rods, for washers, etc.

For convenience in framing and better appearance, the chords are made uniform in section for their entire length.

561. Lower Chord.—Apply formula 54, taking one-half coefficient T on account of splice.

Member Y 2: centre length immaterial; maximum stress = +51.1 tons.

$$A = \frac{2\,W}{T} = \frac{2 \times 51.1}{0.50} = 204 \text{ sq. ins. in cross-section.}$$

And $\frac{204}{12} = 17.0$ ins. deep. Make the lower chord 12 × 18″ for entire length.

562. Web Struts.—Apply Table P, Fig. 583.

Member 2 3:

centre length = $10'\ 9\frac{1}{4}''$; maximum stress = − 5.0; 6 × 6″.

Member 4 5:

centre length $12'\ 9\frac{11}{16}''$; maximum stress = − 6.2; 6 × 6″.

Member 6 7:

centre length $15'\ 7\frac{7}{16}''$; maximum stress = −7.5; 8 × 8″.

Member 8 9:

centre length $18'\ 10\frac{13}{32}''$; maximum stress = − 4.0; 8 × 8″.

563. Web Ties.—Use Table B.

Member 1 2:

length immaterial; maximum stress = + 0.0; 1, 1/2″ rod; ends not upset.

Member 3 4:

maximum stress = + 3.2 tons; 1, 3/4″ rod; ends upset.

Member 5 6:

maximum stress = + 5.0 tons; 1, 15/16″ rod; ends upset.

Member 7 8:

maximum stress = + 12.0 tons; 1, 17/16″ rod; ends upset.

Member 9 9′:

maximum stress = + 20.6 tons; 1, 1 13/16″ rod; ends upset.

Although no stress appears in member 1 2, it actually supports one panel of the lower chord, and a half-inch rod is therefore used for it.

564. Dimension Sheet for Example 1.—

Member.	*P*-stress.	*S*-stress.	*W*-stress.	*Ce*-stress.	*Cr*-stress.	Maximum.	*C*-length.	Dimensions.
X 1	−16.6	−16.6	− 9.0	−14.7	− 8.0	−55.9	10′ 9¼″	12×12″
X 3	−14.3	−14.3	− 7.8	−13.1	− 8.0	−49.7	10′ 9¼″	12×12″
X 5	−12.6	−12.6	− 6.5	−11.4	− 8.0	−44.6	10′ 9¼″	12×12″
X 7	−10.8	−10.8	− 5.3	− 9.8	− 8.0	−39.4	10′ 9¼″	12×12″
X 9	− 9.0	− 9.0	− 4.1	− 8.1	− 5.4	−31.5	10′ 9¼″	12×12″
Y 1	+14.9	+14.9	+ 9.7	+13.7	+ 7.6	+51.1	10′ 0″	12×18″
Y 2	+14.9	+14.9	+ 9.7	+13.7	+ 7.6	+51.1	10′ 0″	12×18″
Y 4	+13.3	+13.3	+ 8.1	+12.2	+ 7.6	+46.4	10′ 0″	12×18″
Y 6	+11.7	+11.7	+ 6.6	+10.6	+ 7.6	+41.6	10′ 0″	12×18″
Y 8	+10.0	+10.0	+ 5.0	+ 9.1	+ 7.6	+36.7	10′ 0″	12×18″
1 2	+ 0.0	+ 0.0	+ 0.0	+ 0.0	+ 0.0	+ 0.0	4′ 0″	1, ½″ R
3 4	+ 0.7	+ 0.7	+ 0.6	+ 1.8	+ 0.0	+ 3.2	8′ 0″	1, ¾″ R
5 6	+ 1.3	+ 1.3	+ 1.3	+ 2.4	+ 0.0	+ 5.0	12′ 0″	1, $\frac{15}{16}$″
7 8	+ 2.0	+ 2.0	+ 1.9	+ 3.0	+ 5.0	+12.0	16′ 0″	1, 1$\frac{7}{16}$″
9 9′	+ 5.3	+ 5.3	+ 2.6	+ 6.0	+ 4.0	+20.6	20′ 0″	1, 1$\frac{13}{16}$″
2 3	− 1.7	− 1.7	− 1.7	− 1.6	− 0.0	− 5.0	10′ 9¼″	6×6″
4 5	− 2.1	− 2.1	− 2.0	− 2.0	− 0.0	− 6.2	12′ 9$\frac{11}{16}$″	6×6″
6 7	− 2.6	− 2.6	− 2.4	− 2.3	− 0.0	− 7.5	15′ 7$\frac{7}{16}$″	8×8″
8 9	− 3.1	− 3.1	− 3.0	− 2.9	− 4.9	−14.0	18′10$\frac{13}{32}$″	8×8″

565. Example 2. A Wooden Truss.—This truss is similar to that of Example 1, excepting that the ceiling and crane loads are

omitted, the truss only being required to support $P + S$ or $P + W$ loads at any time.

Sheathing, rafters and purlins are unchanged from Example 1.

566. Upper Chord.—

Member X 1:

C length 10′ $9\frac{1}{4}$″; maximum stress $-33.2\ T$; $10 \times 10''$.

Same section for entire length.

567. Lower Chord.—

Member Y 1:

maximum stress $+ 29.8\ T$; $10 \times 12''$ for entire length of chord.

568. Web Struts.—

Member 2 3:

C length 10′ $9\frac{1}{4}$″; maximum stress $- 3.4\ T$; $6 \times 6''$

Member 4 5:

C length 12′ $9\frac{11}{16}$″; maximum stress $- 4.2\ T$; $6 \times 6''$.

Member 6 7:

C length 15′ $7\frac{7}{16}$″; maximum stress $- 5.2\ T$; $6 \times 8''$.

Member 8 9:

C length 18′ $10\frac{13}{32}$″; maximum stress $- 6.2\ T$; $8 \times 8''$.

569. Web Ties.—

Member 1 2:

maximum stress $+ 0.0\ T$; 1, $\frac{1}{2}$″ rod; ends not upset.

Member 3 4:

maximum stress $+ 2.0\ T$; 1, $\frac{1}{2}$″ rod; ends upset.

Member 5 6;

maximum stress $+ 3.9\ T$; 1, $\frac{11}{16}$″ rod; ends upset.

Member 7 8:

maximum stress $+ 5.9\ T$; 1, $\frac{13}{16}$″ rod; ends upset.

Member 9 9′:

maximum stress $+ 13.2\ T$; 1, $1\frac{5}{16}$″ rod; ends upset.

Although no stress appears in the member 1 2, it actually supports one-half the weight of two panels of the lower chord, so that it is made $\frac{1}{2}''$.

The dimensions just obtained for the members are next entered on the dimension sheet.

570. Dimension Sheet for Example 2.—

Member.	*P*-stress.	*S*-stress.	*W*-stress.	Maximum.	*C*-length.	Dimensions.
X 1	−16.6	−16.6	− 9.0	−33.2	10′ $9\frac{1}{4}''$	10×10″
X 3	−14.3	−14.3	− 7.8	−28.6	10′ $9\frac{1}{4}''$	10×10″
X 5	−12.6	−12.6	− 6.5	−25.2	10′ $9\frac{1}{4}''$	10×10″
X 7	−10.8	−10.8	− 5.3	−21.6	10′ $9\frac{1}{4}''$	10×10″
X 9	− 9.0	− 9.0	− 4.1	−18.0	10′ $9\frac{1}{4}''$	10×10″
Y 1	+14.9	+14.9	+ 9.7	+29.8	10′ 0″	10×12″
Y 2	+14.9	+14.9	+ 9.7	+29.8	10′ 0″	10×12″
Y 4	+13.3	+13.3	+ 8.1	+26.6	10′ 0″	10×12″
Y 6	+11.7	+11.7	+ 6.6	+23.4	10′ 0″	10×12″
Y 8	+10.0	+10.0	+ 5.0	+20.0	10′ 0″	10×12″
1 2	+ 0.0	+ 0.0	+ 0.0	+ 0.0	4′ 0″	1, $\frac{1}{2}''$ R
3 4	+ 0.7	+ 0.7	+ 0.6	+ 1.4	8′ 0″	1, $\frac{1}{2}''$ R
5 6	+ 1.3	+ 1.3	+ 1.3	+ 2.6	12′ 0″	1, $\frac{11}{16}''$ R
7 8	+ 2.0	+ 2.0	+ 1.9	+ 4.0	16′ 0″	1, $\frac{13}{16}''$ R
9 9′	+ 5.3	+ 5.3	+ 2.6	+10.6	20′ 0″	1, $1\frac{5}{16}''$ R
2 3	− 1.7	− 1.7	− 1.7	− 3.4	10′ $9\frac{1}{4}''$	6×6″
4 5	− 2.1	− 2.1	− 2.0	− 4.2	12′ $9\frac{11}{16}''$	6×6″
6 7	− 2.6	− 2.6	− 2.4	− 5.2	15′ $7\frac{7}{16}''$	6×8″
8 9	− 3.1	− 3.1	− 3.0	− 6.2	18′ $10\frac{13}{32}''$	8×8″

571. Example 3. A Steel Truss.—Resume Example 2 of this chapter and construct the same truss entirely of steel angles. Same as Example 2 of Chapter IV.

Sheathing, rafters, and purlins are unchanged.

572. Changes in P Apex Loads and P Stresses.—The steel truss is somewhat lighter, and it is easily computed, so that the apex weight of truss = 1.310 ton for wooden and 1.249 ton for steel. Hence the *P* stresses in the last dimension sheet will be reduced in the proportion of 1.310 : 1.249. These reductions are easily made by a slide rule and the revised *P* stresses are inserted in the dimension sheet for this example. The *S* and *W* stresses are unchanged.

573. General Remarks on Its Construction.—Each member of this steel truss is usually composed of two angles with unequal legs,

the wider leg being set parallel to the middle vertical plane of the truss.

Each angle is assumed to be riveted through both legs, since this makes the lightest and most economical truss. Rivets $\frac{3}{4}$ inch diameter are used on trusses of 100 ft. span or less; $\frac{7}{8}$ inch rivets on those of wider span and in heavy framing. Gusset plates are made of uniform thickness in a truss, not more than $\frac{1}{8}''$ less in thickness than the diameter of the rivet, in order to develop its full resistance to shear. In accordance with the Chicago ordinance, resistance of the rivet to shear is taken at 6.0 tons per square inch of area of cross-section for single shear; 12.5 tons per square inch of its bearing area for bearing resistance.

The members of each chord are best made of uniform dimensions, excepting that thickness of legs may be diminished with the stresses, using thin fillers for equalizing, or crimping cover plates to fit the thinner angles.

No deductions for rivet holes are made in compression members; rivet holes are taken $\frac{1}{8}''$ larger than diameter of rivet and are deducted in all tension members, this deduction being already provided for in the tables here arranged for dimensioning such tension members. At least two rivets are to be put in each member at a connection.

All stresses in members are to be transmitted through the rivets, cover, and gusset plates at its ends, never by direct contact at ends of compression members.

Trusses must be designed and built in suitable parts for shipment and erection; later being connected by field rivets at the building.

Rivets are to be staggered in the opposite legs to permit convenient shop riveting. Chain riveting is preferable, whenever possible, for two rows of rivets in one leg.

574. Upper Chord.—Use Tables M, N, and O.

Member X 1:

C length $= 10'\ 9\frac{1}{4}''$; max. stress $= -\ 32.4\ T$; $2, 5 \times 3 \times \frac{7}{16}''$.

Member X 3:

C length = $10'\ 9\frac{1}{4}''$; max. stress = $-$ 27.9 T; 2, $5 \times 3 \times \frac{7}{16}''$.

Member X 5:

C length = $10'\ 9\frac{1}{4}''$; max. stress = $-$ 24.6 T; 2, $5 \times 3 \times \frac{3}{8}''$.

Member X 7:

C length = $10'\ 9\frac{1}{4}''$; max. stress = $-$ 21.1 T; 2, $5 \times 3 \times \frac{3}{8}''$.

Member X 9:

C length = $10'\ 9\frac{1}{4}''$; max. stress = $-$ 17.6 T; 2, $5 \times 3 \times \frac{5}{16}''$.

575. Lower Chord.—Use Table G. Avoid the use of special angles marked *.

Member Y 1: max. stress = + 29.1 T; 2, $5 \times 3 \times \frac{3}{8}''$.

Member Y 2: max. stress = + 29.1 T; 2, $5 \times 3 \times \frac{3}{8}''$.

Member Y 4: max. stress = + 26.0 T; 2, $5 \times 3 \times \frac{3}{8}''$.

Member Y 6: max. stress = + 22.9 T; 2, $5 \times 3 \times \frac{3}{8}''$.

Member Y 8: max. stress = + 19.5 T; 2, $5 \times 3 \times \frac{5}{16}''$.

576. Web Struts.—Use Tables M, N, and O.

Member 2 3:

C length = $10'\ 9\frac{1}{4}''$; stress = $-$ 3.4 T; 2, $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}''$.

Member 4 5:

C length = $12'\ 9\frac{11}{16}''$; stress = $-$ 4.1 T; 2, $4 \times 3 \times \frac{5}{16}''$.

Member 6 7:

C length = $15'\ 7\frac{7}{16}''$; stress = $-$ 5.1 T; 2, $5 \times 3\frac{1}{2} \times \frac{5}{16}''$.

Member 8 9:

C length = $18'\ 10\frac{13}{32}''$; stress = $-$ 6.1 T; 2, $6 \times 4 \times \frac{3}{8}''$.

577. Web Ties.—Use Table G. Avoid special angles marked *.

Member 1 2: maximum stress $= +\ 0.0\ T;\ 2,\ 2 \times 2 \times \frac{3}{16}''$.

Member 3 4: maximum stress $= +\ 1.4\ T;\ 2,\ 2 \times 2 \times \frac{3}{16}''$.

Member 5 6: maximum stress $= +\ 2.6\ T;\ 2,\ 2 \times 2 \times \frac{3}{16}''$.

Member 7 8: maximum stress $= +\ 3.9\ T;\ 2,\ 2 \times 2 \times \frac{3}{16}''$.

Member 9 9′: maximum stress $= +\ 10.3\ T;\ 2,\ 2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}''$.

578. Single Shear Rivets in One End of Member.—Use Table T, and insert proper number in dimension sheet.

579. Dimension Sheet for Example 3 Constructed in Steel.—

Member.	P-stress.	S-stress.	W-stress.	Maximum.	C-length.	Dimensions.	S.S. rivets in end.
X 1	−15.8	−16.6	− 9.0	−32.4	10′ 9¼″	2, 5×3×$\frac{7}{16}$″	13
X 3	−13.6	−14.3	− 7.8	−27.9	10′ 9¼″	2, 5×3×$\frac{7}{16}$″	11
X 5	−12.0	−12.6	− 6.5	−24.6	10′ 9¼″	2, 5×3×⅜″	10
X 7	−10.3	−10.8	− 5.3	−21.1	10′ 9¼″	2, 5×3×⅜″	8
X 9	− 8.6	− 9.0	− 4.1	−17.6	10′ 9¼″	2, 5×3×$\frac{5}{16}$″	7
Y 1	+14.2	+14.9	+ 9.7	+29.1	10′ 0″	2, 5×3×⅜″	11
Y 2	+14.2	+14.9	+ 9.7	+29.1	10′ 0″	2, 5×3×⅜″	10
Y 4	+12.7	+13.3	+ 8.1	+26.0	10′ 0″	2, 5×3×⅜″	9
Y 6	+11.2	+11.7	+ 6.6	+22.9	10′ 0″	2, 5×3×⅜″	8
Y 8	+ 9.5	+10.0	+ 5.0	+19.5	10′ 0″	2, 5×3×$\frac{5}{16}$″	8
1 2	+ 0.0	+ 0.0	+ 0.0	+ 0.0	0′ 4″	2, 2×2×$\frac{3}{16}$″	2
3 4	+ 0.7	+ 0.7	+ 0.6	+ 1.4	8′ 0″	2, 2×2×$\frac{3}{16}$″	2
5 6	+ 1.2	+ 1.3	+ 1.3	+ 2.6	12′ 0″	2, 2×2×$\frac{3}{16}$″	2
7 8	+ 1.9	+ 2.0	+ 1.9	+ 3.9	16′ 0″	2, 2×2×$\frac{3}{16}$″	3
9 9′	+ 5.0	+ 5.3	+ 2.6	+10.3	20′ 0″	2, 2½×2½×¼″	5
2 3	− 1.6	− 1.7	− 1.7	− 3.4	10′ 9¼″	2, 3½×2½×¼″	2
4 5	− 2.0	− 2.1	− 2.0	− 4.1	12′ 9$\frac{11}{16}$″	2, 4×3×$\frac{5}{16}$″	2
6 7	− 2.5	− 2.6	− 2.4	− 5.1	15′ 7$\frac{7}{16}$″	2, 5×3½×$\frac{5}{16}$″	2
8 9	− 3.0	− 3.1	− 3.0	− 6.1	18′ 10$\frac{13}{32}$″	2, 6×4×⅜″	3

580. Example 5.—A steel Fink truss.

581. General Remarks.—Resume Example 5 of Chapter IV. This is a Fink truss of 128 ft. span with 16 panels; rise of upper chord 25.6 ft.; of lower chord at middle 2.0 ft. This truss will require expansion rolls at one end. Three different methods of

construction are worked out here for comparison of their economy in Chapter XIII.

582. A.—Upper chord composed of 2 latticed channels spaced apart to make it equally stiff in both directions. Web struts are each made of a pair of angles riveted to a $\frac{5}{8}''$ gusset eye-plate at each end with hole for pin. Lower chord and web ties are each composed of a pair of round steel rods with loop-welded eyes at ends, flattened to $\frac{4}{5}''$ diameter. Pin connections at all apexes of the truss.

583. B.—Each member is composed of a pair of angles with $\frac{5}{8}''$ gusset plates and riveted connections at all apexes.

584. C.—Upper chord consists of 2 channels riveted directly to $\frac{5}{8}''$ gusset plates; all other members are composed of a pair of angles; all connections at apexes to be riveted.

585. Sheathing.—White pine $\frac{7}{8}''$ matched sheathing covered by painted tin roof.

$w' = \cos 21.8° \ (2\# + 3\# + 25\# \ \cos 21.8°) = 26.19\#$ per sq. ft. normal.

$w'' = \sin 21.8° \ (5\# + 25\# \ \cos 21.8°) = 10.48\#$ per sq. ft. parallel.

w'' is resisted by edgewise stiffness of sheathing and may be neglected.

Apply formulas 107 and 110.

$$L = 51.6\,t\sqrt{\frac{F}{w'}} = 51.6 \times .875\sqrt{\frac{0.45}{26.19}} = 5.62 \text{ ft. on centres of rafters.}$$

$$L = 1.44\,t\sqrt[3]{\frac{E}{w'}} = 1.44 \times .875\sqrt[3]{\frac{500}{26.19}} = 3.37 \text{ ft. on centres of rafters.}$$

Then space rafters 3 ft. on centres.

586. Rafters.—Each a steel channel (Art. 469).

$w' = \cos 21.8° \ (2 + 3 + 4 + 25 \cos 21.8°) = 29.92\#$ per sq. ft. of roof.

$w'' = \sin 21.8° \ (2 + 3 + 4 + 25 \cos 21.8°) = 11.96\#$ per sq. ft. of roof.

Apply formulas 97 and 101 for steel rafters.

$$\frac{I}{c} = \frac{w' L^2 e}{128000} = \frac{29.92 \times 8.62^2 \times 36}{128000} = 0.63.$$

$$I = \frac{w' L^3 e}{515556} = \frac{29.92 \times 8.62^3 \times 36}{515556} = 1.34.$$

Since $\dfrac{I}{c} = 1.1$ and $I = 1.6$ for a 3 in. 4# channel, this will suffice for w'.

Longitudinal compression at mid-length of rafter $= \dfrac{11.96 \times 8.62 \times 3}{2 \times 2000}$ $= 0.0773$ ton.

Then $\dfrac{0.0773}{1.19} = 0.065$ ton per sq. inch = uniform longitudinal compression.

$W' = \dfrac{8.62 \times 3 \times 29.92}{2000} = 0.387$ ton = total normal load on rafter.

$\Delta = \dfrac{22.5\ W'L^3}{E\,I} = \dfrac{22.5 \times 0.387 \times 8.62^3}{14500 \times 1.6} = 0.241$ inch deflection at middle. Then $.0773\left(1 + \dfrac{6\,\Delta}{d}\right) = .0773\left(1 + \dfrac{6 \times 0.241}{3}\right) =$ 0.1146 ton per sq. inch = maximum fibre stress at mid-length of rafter (Art. 470).

And $8.00 - 0.1146 = 7.8854$ = maximum safe value of F for rafter.

Apply formula 77 with this value for F.

$$\frac{I}{c} = \frac{3\,W'\,L}{2\,F} = \frac{3 \times 0.387 \times 8.62}{2 \times 7.8854} = 0.635.$$

Since $\dfrac{I}{c} = 1.1$ for a 3″ 4# channel, this will be amply safe to resist both the normal component W' and the parallel component W'' acting on the rafter.

587. Purlins.—Composed of 2 steel channels, latticed together (Art. 472).

$w' = \cos 21.8° \,(2 + 3 + 4 + 3 + 25 \cos 21.8°) = 33.47$# per sq. ft. of roof.

$w'' = \sin 21.8° \,(2 + 3 + 4 + 3 + 25 \cos 21.8°) = 13.07$# per sq. ft. of roof.

$W' = A\,w' = 137.92 \times 33.47 = 4616$# $= 2.308$ tons = normal component.

$W'' = A\,w'' = 137.92 \times 13.07 = 1803$# $= 0.902$ ton = parallel component.

Apply formulas 83 and 86 (Art. 471).

For normal component of load on purlin:

$$\frac{I}{c} = \frac{3\,W'\,L}{16} = \frac{3 \times 2.308 \times 16}{16} = 6.92.$$

$$I = .0466\,W'\,L^2 = .0466 \times 2.308 \times 16^2 = 27.6.$$

For parallel component of load on purlin:

$$\frac{I}{c} = \frac{3\,W''\,L}{16} = \frac{3 \times 0.902 \times 16}{16} = 2.70.$$

$$I = .0466\,W''\,L^2 = .0466 \times 0.902 \times 16^2 = 10.8.$$

Since 2 channels compose each purlin, each channel is required to have half the values of $\frac{I}{c}$ and of I corresponding to W'. Hence two, 7″ 9.75 # channels will suffice for the purlin. According to Table J, they should be spaced 4 1/4″ apart to have equal stiffness in both directions.

Formula 145 should then be applied to determine whether the maximum fibre stress exceeds the safe limit of 8.00 tons per sq. inch (Art. 476).

$$.75\,L\left(\frac{W'\,d}{I_y} + \frac{W''\,b}{I_x}\right) =$$

$$.75 \times 16\left(\frac{2.308 \times 7}{2 \times 21.1} + \frac{0.902 \times (4.25 + 2 \times 2.09)}{2 \times 21.1}\right) =$$

6.71 tons per sq. inch. Hence this purlin is amply safe.

588. Calculation of Exact Lengths of Truss Members.—These lengths are computed in the same manner as those of the Fink truss treated in Chapter VII, from which these dimensions are taken.

$$\text{Total length of principal} = \sqrt{64^2 + 25.6^2} = 68'\,11\frac{3}{32}''.$$

$$\text{Half length of principal} = 34' = 34'\,5\frac{9}{16}''.$$

$$\text{Eighth length of principal} = 8'\,7\frac{3}{8}''.$$

$$\text{Total length of half lower chord} = \sqrt{64^2 + 2^2} = 64'\,0\frac{3}{8}''.$$

$$\text{Tan } i \text{ for principal} = \frac{25.6}{64.0} = \tan 21^\circ\,48'\,5''.$$

$$\text{Tan } i' \text{ for lower chord} = \frac{2.0}{64.0} = \tan 1^\circ\,47'\,24''.$$

And i'' between principal and lower chord = 21° 48′ 5″ − 1° 47′ 24″ = 20° 0′ 41″.

Then length of 7 8 = tan 20° 0′ 41″ × 34′ $5\frac{9}{16}''$ = 12′ $6\frac{17}{32}''$.

And 3 4 = 11 12 = half 7 8 = 6′ $3\frac{9}{32}''$.

1 2 = 5 6 = 9 10 = 13 14 = one-fourth 7 8 = 3′ $1\frac{5}{8}''$.

Length of Y 1 + Y 3 + Y 7 =

$$\sqrt{(34'\,5\,9/16'')^2+(12'\,6\,17/32'')^2} = 36'\,8\frac{3}{32}''.$$

Y 7 = half 36′ $8\frac{3}{32}''$ = 18′ $4\frac{1}{16}''$.

Y 1 = Y 3 = fourth 36′ $8\frac{3}{32}''$ = 9′ $2\frac{1}{32}''$.

Y 15 = 64′ $0\frac{3}{8}''$ − 36′ $8\frac{3}{32}'$ = 27′ $4\frac{9}{32}''$.

15 15 = 25.6′ − 2′ = 23.6′ = 23′ $7\frac{3}{16}''$.

Y 1 = 2 3 = 4 5 = 4 7 = 6 7 = 8 9 = 8 11 = 10 11 = 12 13 = 12 15 = 14 15 = 9′ $2\frac{1}{32}''$.

Y 7 = 8 15 = 18′ $4\frac{1}{16}''$.

589. Upper Chord.—Sections of the members are determined by Table J. Assuming that only one splice is made at middle of each principal, we obtain the following:

Member X 1: C length = 8′ $7\frac{3}{8}''$; max. stress − 60.8 tons; 2, 9″ 15# channels, spaced at least $5\frac{1}{2}$ ins. apart and latticed on flanges. Uniform cross-section for entire length of upper chord.

590. Lower Chord.—Composed of pairs of loop-welded eye-rods.

Apply Table B to determine diameters of rods.

Member Y 1:

max. stress + 56.5 tons; 2, $2\frac{1}{2}''$ rods; ends flatted to 2″.

Member Y 3:

max. stress + 52.7 tons; 2, $2\frac{3}{8}''$ rods; ends flatted to $1\frac{7}{8}''$.

Member Y 7:

max. stress + 44.9 tons; 2, $2\frac{1}{4}''$ rods; ends flatted to $1\frac{3}{4}''$.

Member Y 15:

max. stress + 29.7 tons; 2, $1\frac{13}{16}''$ rods; ends flatted to $1\frac{7}{16}''$.

591. Web Struts.—Apply Tables M, N, and O; angles being riveted through wide legs only.

Member 1 2:

$C.L. = 3'\ 1\frac{5}{8}''$; stress − 2.6 tons; 2, $2 \times 2 \times \frac{3}{16}''$ Ls.

Member 3 4:

$C.L. = 6'\ 3\frac{9}{32}''$; stress − 5.2 tons; 2, $2\frac{1}{2} \times 2 \times \frac{1}{4}''$ Ls.

Member 5 6:

$C.L. = 3'\ 1\frac{5}{8}''$; stress − 2.6 tons; 2, $2 \times 2 \times \frac{3}{16}''$ Ls.

Member 7 8:

$C.L. = 12'\ 6\frac{17}{32}''$; stress − 10.4 tons; 2, $4 \times 3 \times \frac{5}{16}''$ Ls.

Member 9 10:

$C.L. = 3'\ 1\frac{5}{8}''$; stress = − 2.6 tons; 2, $2 \times 2 \times \frac{3}{16}''$ Ls.

Member 11 12:

$C.L. = 6'\ 3\frac{9}{32}''$; stress = − 5.2 tons; 2, $2\frac{1}{2} \times 2 \times \frac{1}{4}''$ Ls.

Member 13 14:

$C.L. = 3'\ 1\frac{5}{8}''$; stress = − 2.6 tons; 2, $2 \times 2 \times \frac{3}{16}''$ Ls.

Apply Table T to determine number of single shear $\frac{3}{4}''$ rivets required in each end of each web strut. Wide leg only is riveted to $\frac{5}{8}''$ gusset plate with hole to receive pin at connection.

592. Web Ties.—Apply Table B again to determine diameters of rods.

Member 2 3:

stress = + 3.8 tons; 2, $\frac{11}{16}''$ rods; ends flatted to $\frac{9}{16}''$.

Member 4 5:

stress = + 3.8 tons; 2, $\frac{11}{16}''$ rods; ends flatted to $\frac{9}{16}''$.

Member 4 7:

stress = + 7.8 tons; 2, 1'' rods; ends flatted to $\frac{13}{16}''$.

Member 6 7:

stress = + 11.6 tons; 2, $1\frac{3}{16}''$ rods; ends flatted to $\frac{15}{16}''$.

Member 8 9:

stress = + 11.6 tons; 2, $1\frac{3}{16}''$ rods; ends flatted to $\frac{15}{16}''$.

Member 8 11:

stress = + 7.8 tons; 2, 1'' rods; ends flatted to $\frac{13}{16}''$.

Member 8 15:

stress = + 15.3 tons; 2, $1\frac{5}{16}''$ rods; ends flatted to $1\frac{1}{16}''$.

Member 10 11:

stress = + 3.8 tons; 2, $\frac{11}{16}''$ rods; ends flatted to $\frac{9}{16}''$.

Member 12 13:

stress = + 3.8 tons; 2, $\frac{11}{16}''$ rods; ends flatted to $\frac{9}{16}''$.

Member 12 15:

stress = + 23.1 tons; 2, $1\frac{5}{8}''$ rods; ends flatted to $1\frac{5}{16}''$.

Member 14 15:

stress = + 26.8 tons; 2, $1\frac{3}{4}''$ rods; ends flatted to $1\frac{3}{8}''$.

Member 15 15:

stress = + 1.9 tons; 1, $\frac{11}{16}''$ rod; ends flatted to $\frac{9}{16}''$.

These results are collected on the dimension sheet for this Example (5 A).

593. Dimension Sheet for Example 5 A.—

Member.	*P*-stress.	*S*-stress.	*W*-stress.	Maximum.	*C*-length.	Dimensions.	Ends.
X 1	−25.7	−35.1	−14.7	−60.8	$8' 7\frac{3}{8}''$	2, 9″ 15 # C.	......
X 2	−25.3	−34.5	−14.7	−59.8	$8' 7\frac{3}{8}''$	2, 9″ 15 # C.	......
X 5	−24.9	−33.9	−14.7	−58.8	$8' 7\frac{3}{8}''$	2, 9″ 15 # C.	......
X 6	−24.4	−33.2	−14.7	−57.6	$8' 7\frac{3}{8}''$	2, 9″ 15 # C.	......
X 9	−24.0	−32.7	−14.7	−56.7	$8' 7\frac{3}{8}''$	2, 9″ 15 # C.	......
X 10	−23.5	−32.1	−14.7	−55.6	$8' 7\frac{3}{8}''$	2, 9″ 15 # C.	......
X 13	−23.1	−31.5	−14.7	−54.6	$8' 7\frac{3}{8}''$	2, 9″ 15 # C.	......
X 14	−22.7	−30.9	−14.7	−53.6	$8' 7\frac{3}{8}''$	2, 9″ 15 # C.	......
Y 1	+23.9	+32.6	+16.4	+56.5	$9' 2\frac{1}{32}''$	$2, 2\frac{1}{2}''$ R.	2″
Y 3	+22.3	+30.4	+15.4	+52.7	$9' 2\frac{1}{32}''$	$2, 2\frac{3}{8}''$ R.	$1\frac{7}{8}''$
Y 7	+19.0	+25.9	+12.2	+44.9	$18' 4\frac{1}{16}''$	$2, 2\frac{1}{4}''$ R.	$1\frac{3}{4}''$
Y 15	+12.7	+17.0	+ 6.2	+29.7	$27' 4\frac{9}{32}''$	$2, 1\frac{13}{16}''$ R.	$1\frac{7}{16}''$
1 2	− 1.1	− 1.5	− 1.0	− 2.6	$3' 1\frac{5}{8}''$	$2, 2\times2\times\frac{3}{16}''$ L.	3 rivets
3 4	− 2.2	− 3.0	− 2.0	− 5.2	$6' 3\frac{9}{32}''$	$2, 2\frac{1}{2}\times2\times\frac{1}{4}''$ L.	4 rivets
5 6	− 1.1	− 1.5	− 1.0	− 2.6	$3' 1\frac{5}{8}''$	$2, 2\times2\times\frac{3}{16}''$ L.	3 rivets
7 8	− 4.3	− 6.1	− 4.1	−10.4	$12' 6\frac{17}{32}''$	$2, 4\times3\times\frac{5}{16}''$ L.	4 rivets
9 10	− 1.1	− 1.5	− 1.0	− 2.6	$3' 1\frac{5}{8}''$	$2, 2\times2\times\frac{3}{16}''$ L.	3 rivets
11 12	− 2.2	− 3.0	− 2.0	− 5.2	$6' 3\frac{9}{32}''$	$2, 2\frac{1}{2}\times2\times\frac{1}{4}''$ L.	4 rivets
13 14	− 1.1	− 1.5	− 1.0	− 2.6	$3' 1\frac{5}{8}''$	$2, 2\times2\times\frac{3}{16}''$ L.	3 rivets
2 3	+ 1.6	+ 2.2	+ 1.5	+ 3.8	$9' 2\frac{1}{32}''$	$2, \frac{11}{16}''$ R.	$\frac{9}{16}''$
4 5	+ 1.6	+ 2.2	+ 1.5	+ 3.8	$9' 2\frac{1}{32}''$	$2, \frac{11}{16}''$ R.	$\frac{9}{16}''$
4 7	+ 3.3	+ 4.5	+ 2.9	+ 7.8	$9' 2\frac{1}{32}''$	2, 1″ R.	$\frac{13}{16}''$
6 7	+ 4.9	+ 6.7	+ 4.4	+11.6	$9' 2\frac{1}{32}''$	$2, 1\frac{3}{16}''$ R.	$\frac{15}{16}''$
8 9	+ 4.9	+ 6.7	+ 4.4	+11.6	$9' 2\frac{1}{32}''$	$2, 1\frac{3}{16}''$ R.	$\frac{15}{16}''$
8 11	+ 3.3	+ 4.5	+ 2.9	+ 7.8	$9' 2\frac{1}{32}''$	2, 1″ R.	$\frac{13}{16}''$
8 15	+ 6.3	+ 9.0	+ 6.0	+15.3	$18' 4\frac{1}{16}''$	$2, 1\frac{5}{16}''$ R.	$1\frac{1}{16}''$
10 11	+ 1.6	+ 2.2	+ 1.5	+ 3.8	$9' 2\frac{1}{32}''$	$2, \frac{11}{16}''$ R.	$\frac{9}{16}''$
12 13	+ 1.6	+ 2.2	+ 1.5	+ 3.8	$9' 2\frac{1}{32}''$	$2, \frac{11}{16}''$ R.	$\frac{9}{16}''$
12 15	+ 9.6	+13.5	+ 8.9	+23.1	$9' 2\frac{1}{32}''$	$2, 1\frac{5}{8}''$ R.	$1\frac{5}{16}''$
14 15	+11.2	+15.6	+10.4	+26.8	$9' 2\frac{1}{32}''$	$2, 1\frac{3}{4}''$ R.	$1\frac{3}{8}''$
15 15′	+ 0.9	+ 1.0	+ 0.5	+ 1.9	$23' 7\frac{3}{16}''$	$1, \frac{11}{16}''$ R.	$\frac{9}{16}''$

594. B.—Truss members entirely composed of pair of angles with gussets and riveted connections at all apexes. Both legs of angles riveted.

Sheathing, rafters, and purlins are unchanged from Example 4 A.

595. Upper Chord.—Apply Table N for determining sections of members.

Member X 1: stress $= -60.8\,T$; $C.L.\ 8'\,7\frac{3}{8}''$; $2,\ 6\times4\times\frac{9}{16}''$ Ls.

Member X 2: stress $= -59.8\ T$; $C.L.\ 8'\ 7\frac{3}{8}''$; $2, 6 \times 4 \times \frac{9}{16}''$ Ls.

Member X 5: stress $= -58.8\ T$; $C.L.\ 8'\ 7\frac{3}{8}''$; $2, 6 \times 4 \times \frac{9}{16}''$ Ls.

Member X 6: stress $= -57.6\ T$; $C.L.\ 8'\ 7\frac{3}{8}''$; $2, 6 \times 4 \times \frac{9}{16}''$ Ls.

Member X 9: stress $= -56.7\ T$; $C.L.\ 8'\ 7\frac{3}{8}''$; $2, 6 \times 4 \times \frac{1}{2}''$ Ls.

Member X 10: stress $= -55.6\ T$; $C.L.\ 8'\ 7\frac{3}{8}''$; $2, 6 \times 4 \times \frac{1}{2}''$ Ls.

Member X 13: stress $= -54.6\ T$; $C.L.\ 8'\ 7\frac{3}{8}''$; $2, 6 \times 4 \times \frac{1}{2}''$ Ls.

Member X 14: stress $= -53.6\ T$; $C.L.\ 8'\ 7\frac{3}{8}''$; $2, 6 \times 4 \times \frac{1}{2}''$ Ls.

Upper chord is spliced at middle apex of principal only.

596. Lower Chord.—Apply Table G for determining sections of members.

Member Y 1: stress $= +56.5\ T$; $2, 6 \times 3\frac{1}{2} \times \frac{9}{16}''$ Ls.

Member Y 3: stress $= +52.7\ T$; $2, 6 \times 3\frac{1}{2} \times \frac{9}{16}''$ Ls.

Member Y 7: stress $= +44.9\ T$; $2, 6 \times 3\frac{1}{2} \times \frac{7}{16}''$ Ls.

Member Y 15: stress $= +29.7\ T$; $2, 5 \times 3\frac{1}{2} \times \frac{7}{16}''$ Ls.

Lower chord is spliced at 5 apexes as shown in details.

597. Web Struts.—Apply Tables M, N, and O, as required.

Member 1 2:

stress $= -2.6\ T$; $C.L.\ 3'\ 1\frac{5}{8}''$; $2, 2 \times 2 \times \frac{3}{16}''$ Ls.

Member 3 4:

stress $= -5.2\ T$; $C.L.\ 6'\ 3\frac{9}{32}''$; $2, 2\frac{1}{2} \times \frac{3}{16}''$ Ls.

Member 5 6:

stress $= -2.6\ T$; $C.L.\ 3'\ 1\frac{5}{8}''$; $2, 2 \times 2 \times \frac{3}{16}''$ Ls.

Member 7 8:

stress $= -$ 10.4 tons; $C.L.\ 12'\ 6\frac{17}{32}''$; $2, 4 \times 3 \times \frac{5}{16}''$ Ls.

Member 9 10:

stress = $-$ 2.6 tons; *C.L.* $3'\ 1\frac{5}{8}''$; $2, 2 \times 2 \times \frac{3}{16}''$ Ls.

Member 11 12:

stress = $-$ 5.2 tons; *C.L.* $6'\ 3\frac{9}{32}''$; $2, 2\frac{1}{2} \times 2 \times \frac{3}{16}''$ Ls.

Member 13 14:

stress = $-$ 2.6 tons; *C.L.* $3'\ 1\frac{5}{8}''$; $2, 2 \times 2 \times \frac{3}{16}''$ Ls.

598. Web Ties.—Apply Table G.

Member 2 3: stress = + 3.8 tons; $2, 2 \times 2 \times \frac{3}{16}''$ Ls.

Member 4 5: stress = + 3.8 tons; $2, 2 \times 2 \times \frac{3}{16}''$ Ls.

Member 4 7: stress = + 7.8 tons; $2, 2\frac{1}{2} \times 2 \times \frac{1}{4}''$ Ls.

Member 6 7: stress = + 11.6 tons; $2, 2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}''$ Ls.

Member 8 9: stress = + 11.6 tons; $2, 2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}''$ Ls.

Member 8 11: stress = + 7.8 tons; $2, 2\frac{1}{2} \times 2 \times \frac{1}{4}''$ Ls.

Member 8 15: stress = + 15.3 tons; $2, 3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}''$ Ls.

Member 10 11: stress = + 3.8 tons; $2, 2 \times 2 \times \frac{3}{16}''$ Ls.

Member 12 13: stress = + 3.8 tons; $2, 2 \times 2 \times \frac{3}{16}''$ Ls.

Member 12 15: stress = + 23.1 tons; $2, 3\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{8}''$ Ls.

Member 15 15: stress = + 26.8 tons; $2, 3\frac{1}{2} \times 2\frac{1}{2} \times \frac{7}{16}''$ Ls.

Member 15 15: stress = + 1.9 tons; $2, 2 \times 2 \times \frac{3}{16}''$ Ls.

Table T is used for determining the number of single shear rivets to be used in each end of each member of the truss.

The results are collected on the following stress sheet. The columns for *P*, *S*, and *W* stresses are omitted, being the same as in Case A.

599. Dimension Sheet for Example 5 B.—

Member.	Maximum.	*C*-length.	Dimensions.	S. S. rivets.
X 1	−60.8	8′ 7 3/8″	2, 6×4×9/16″	23
X 2	−59.8	8′ 7 3/8″	2, 6×4×9/16″	23
X 5	−58.8	8′ 7 3/8″	2, 6×4×9/16″	23
X 6	−57.6	8′ 7 3/8″	2, 6×4×9/16″	22
X 9	−56.7	8′ 7 3/8″	2, 6×4×1/2″	22
X 10	−55.6	8′ 7 3/8″	2, 6×4×1/2″	21
X 13	−54.6	8′ 7 3/8″	2, 6×4×1/2″	21
X 14	−53.6	8′ 7 3/8″	2, 6×4×1/2″	21
Y 1	+56.5	9′ 2 1/32″	2, 6×3 1/2×9/16″	22
Y 3	+52.7	9′ 2 1/32″	2, 6×3 1/2×9/16″	20
Y 7	+44.9	18′ 4 1/16″	2, 6×3 1/2×7/16″	17
Y 15	+29.7	27′ 4 9/32″	2, 5×3×7/16″	12
1 2	− 2.6	3′ 1 5/8″	2, 2×2×3/16″	2
3 4	− 5.2	6′ 3 9/32″	2, 2 1/2×2×3/16″	4
5 6	− 2.6	3′ 1 5/8″	2, 2×2×3/16″	2
7 8	−10.4	12′ 6 17/32″	2, 4×3×5/16″	4
9 10	− 2.6	3′ 1 5/8″	2, 2×2×3/16″	2
11 12	− 5.2	6′ 3 9/32″	2, 2 1/2×2×3/16″	4
13 14	− 2.6	3′ 1 5/8″	2, 2×2×3/16″	2
2 3	+ 3.8	9′ 2 1/32″	2, 2×2×3/16″	3
4 5	+ 3.8	9′ 2 1/32″	2, 2×2×3/16″	3
4 7	+ 7.8	9′ 2 1/32″	2, 2 1/2×2×1/4″	4
6 7	+11.6	9′ 2 1/32″	2, 2 1/2×2 1/2×1/4″	6
8 9	+11.6	9′ 2 1/32″	2, 2 1/2×2 1/2×1/4″	6
8 11	+ 7.8	9′ 2 1/32″	2, 2 1/2×2×1/4″	4
8 15	+15.3	18′ 4 1/16″	2, 3 1/2×2 1/2×1/4″	7
10 11	+ 3.8	9′ 2 3/32″	2, 2×2×3/16″	3
12 13	+ 3.8	9′ 2 3/32″	2, 2×2×3/16″	3
12 15	+23.1	9′ 2 3/32″	2, 3 1/2×2 1/2×3/8″	9
14 14	+26.8	9′ 2 3/32″	2, 3 1/2×2 1/2×7/16″	11
15 15′	+ 1.9	23′ 7 3/16″	2, 2×2×3/16″	2

600. C.—Upper chord composed of two channels riveted to 5/8″ gussets.

Apply Table L to determine section of upper chord.

Member X 1: stress = − 60.8 T; $C.$ $L.$ $8'\ 7\ \dfrac{3}{8}''$; 2, 10″ 20 # channels.

This section is used for the entire length of each principal. The remaining members of the truss have the dimensions obtained in Case B and entered on the last stress sheet.

The comparative economy of the three methods of construction for this Example 5 will be considered in Chapter XIII. But it appears probable that that used in Case B is simplest and most economical. Since the only difference in Case C from B is in the section of the upper chord, it is not necessary to make out a stress sheet for Case C.

601. Example 10. A Steel Semicircular Crescent Truss.—Resume Example 10 of Chapter IV, modified as follows:

602. General Remarks.—The form of this truss is changed from that shown in Fig. 98, in which the radials are drawn to the centre of upper chord, to that later given in Fig. 432, where both the upper and lower chords are each divided into 14 equal parts, the radials and diagonals then connecting their apexes.

This change in type of truss does not affect the loads, which remain as given in Chapter IV, but it requires new *P*, *S*, and *W* stress diagrams to be drawn, thus slightly changing the stress in the members, which becomes evident by comparing the stress sheet given in Chapter IV with that here given for the modified truss.

Centre lengths of all members are computed for this modified truss in Chapter VII, from which they are here taken.

Since several purlins rest on each member of the upper chord, these members must sustain a bending moment in addition to longitudinal compression.

Two different methods of construction of this truss will be examined.

603. A.—All members of each chord are straight between apexes. This requires a splice to be made at each apex, and the wooden purlins must be set with radial sides and their outer or upper edges in a semicircle to receive the sheathing bent over them, which runs parallel to the upper chord. This is easily arranged by blocking out their ends on the chord, or better by properly boxing the ends down on the chord.

604. B.—All members of the chords are curved to the proper semicircle to directly receive the purlins. This may have a better appearance, but is more troublesome in practice, and the sections of the members must be changed to allow for the moment produced by curvature of the member. Those of the upper chord will be a little lighter, and those of the lower chord slightly heavier, as apparent on examination of the dimension sheets.

605. Sheathing.—Formulas 107 and 110 for the maximum safe span of sheathing may be simplified for this example as follows:

$$51.6\,t\sqrt{\frac{F}{w'}} = 51.6 \times .875\sqrt{\frac{.70}{w'}} = \frac{37.78}{\sqrt{w'}}$$

$$1.44\,t\sqrt[3]{\frac{E}{w'}} = 1.44 \times .875\sqrt[3]{\frac{700}{w'}} = \frac{11.185}{\sqrt[3]{w'}}$$

a. At apex *A*.

$w' = \cos 90° (2 + 4) + 40 = 40$ lbs. per sq. ft.

$$\frac{37.78}{\sqrt{w'}} = \frac{37.78}{\sqrt{40}} = \frac{37.78}{6.32} = 6.00 \text{ ft. on centres of purlins.}$$

$$\frac{11.185}{\sqrt[3]{w'}} = \frac{11.185}{\sqrt[3]{40}} = \frac{11.185}{3.42} = 3.28 \text{ ft. on centres of purlins.}$$

Therefore $\frac{11.247}{3.28} = 3.43$ spaces per panel, say 4.

b. Apex *B*.

$w' = \cos 77.2° (6) + 40 = 41.3$ lbs. per sq. ft. *P* and *W* loads.

$w' = \cos 77.2° (6) + 10 \cos 77.2° = 2.7$ lbs. per sq. ft. *P* and *S*.

The larger value of w' is to be taken.

$$\frac{37.78}{\sqrt{w'}} = \frac{37.78}{\sqrt{41.3}} = \frac{37.78}{6.43} = 5.88 \text{ ft. on centres of purlins.}$$

$$\frac{11.185}{\sqrt[3]{w'}} = \frac{11.185}{\sqrt[3]{41.3}} = \frac{11.185}{3.46} = 3.24 \text{ ft. on centres of purlins.}$$

c. Apex *C*.

$w' = \cos 64.4° (6) + 40 = 42.6$ lbs. per sq. ft.

By formulas, 5.79 and 3.21 ft. on centres of purlins.

d. Apex *D*.

$w' = \cos 51.6° (6) + 40 = 43.7$ lbs. per sq. ft.

By formulas, 5.72 and 3.18 ft. on centres of purlins.

e. Apex *E*.

$$w' = \cos 38.7° \times 6 + \frac{8}{9}\,38.7° = 39.1 \text{ lbs. per sq. ft.}$$

By formulas, 6.05 and 3.30 ft. on centres of purlins.

f. Apex *F*.

$$w' = \cos 25.8° \times 6 + \frac{8}{9}\,25.8° = 28.3 \text{ lbs. per sq. ft.}$$

By formulas, 7.11 and 3.67 ft. on centres of purlins.

This panel might have three spaces, but four would make a better appearance.

g. Apex *G*.

$$w' = \cos 13.0° \times 6 + \frac{8}{9}\ 13.0° = 17.4 \text{ lbs. per sq. ft.}$$

By formulas, 9.07 and 4.32 ft. on centres of purlins.

h. Apex *H*.

$$w' = \cos 0° \times 6 + \frac{8}{9}\ 0.0° = 6.0 \text{ lbs. per sq. ft.}$$

$$w' = \sin 0° \ (6 + 10 \cos 0°) = 16.0 \text{ lbs. per sq. ft.}$$

By formulas, 9.46 and 4.44 ft. on centres of purlins.

606. Purlins.—Since the purlins may be assumed to be about $10''$ deep; $2\frac{1}{4}''$ from centre line of upper chord to its top; $\frac{7}{8}''$ sheathing: this makes the radius of the outer surface of roof $= 50.0' + 10'' + 2\frac{1}{4}'' + \frac{7}{8}'' = 51'\ 1\frac{1}{8}'' = 51.0937'$. Hence with 4 spaces per panel, the surface of roof supported by one purlin $=$ $\frac{51.0937 \times \pi \times 16}{4 \times 14} = 45.86$ sq. ft.

Apply formulas 77 and 80.

Since $\frac{3}{2}$, 675, $L = 16$, $F = .70$, and $E = 700$, are all constant values, these formulas may also be simplified as follows for this example:

$$\frac{I}{c} = \frac{3\,W L}{2\,F} = \frac{3\,W \times 16}{2 \times .7} = 34.29\ W$$

$$I = \frac{675\,W L^2}{E} = \frac{675 \times W \times 16^2}{700} = 246.85\ W.$$

a. Apex *A*.

$$w' = \cos 90° \ (2 + 4 + 4) + 40 = 40 \text{ lbs. per sq. ft.}$$

$$w'' = \sin 90° \ (2 + 4 + 4) = 10 \text{ lbs. per sq. ft.}$$

$$W' = 45.86 \times 40 = 1874 \text{ lbs.} = 0.937 \text{ ton.}$$

$$W'' = 45.86 \times 10 = 459 \text{ lbs.} = 0.230 \text{ ton.}$$

For normal component:

$$\frac{I}{c} = 34.29\ W' = 34.29 \times 0.937 = 32.1.$$

$$I = 246.85\ W' = 246.85 \times 0.937 = 231.3.$$

For parallel component:

$$\frac{I}{c} = 34.29\ W'' = 34.29 \times 0.230 = 7.9.$$

$$I = 246.85\ W'' = 246.85 \times 0.230 = 56.8.$$

Apply Tables R and S.

By Table R for 32.1: 2 × 12, 2 × 10, 3 × 8, 6 × 6.
By Table R for 7.9: 12 × 1, 10 × 1, 8 × 1, 6 × 2.
By Table S for 231.3: 2 × 12, 3 × 10, 6 × 8, 14 × 6.
By Table S for 56.8: 12 × 1, 10 × 1, 8 × 2, 6 × 4.
Hence it appears that 4 × 10 would suffice for purlin.
Check these dimensions by formula 145.

$$.75\ L\left(\frac{W'd}{I_y} = \frac{W''b}{I_x}\right) = .75 \times 16\left(\frac{0.937 \times 10}{333} + \frac{0.230 \times 4}{53}\right) =$$

0.546 ton per sq. in. = maximum fibre stress, less than the limit of 0.700. Hence the purlin is entirely safe.

b. Apex *B*.

$w' = 10 \cos 77.2° + 40 = 42.2$ # per sq. ft.
$w'' = 10 \sin 77.2° = 9.75$ # per sq. ft.
$W' = 45.86 \times 42.22 = 1937$ # $= 0.969$ ton.
$W'' = 45.89 \times 9.75 = 447$ # $= 0.224$ ton.

Applying the same formulas and tables, 4 × 10 is found sufficient, with a maximum fibre stress of 0.552 ton per sq. in.

c. Apex *C*.

$w' = 10 \cos 64.4° + 40 = 44.32$ # per sq. ft.
$w'' = 10 \sin 64.4° = 9.02$ # per sq. ft.

Applying the same formulas and tables, the maximum fibre stress in 4 × 10 purlin is found = 0.553 ton per sq. inch.

d. Apex *D*.

$w' = 10 \cos 51.6° + 40 = 46.21$ # per sq. ft.
$w'' = 10 \sin 51.6° = 7.84$ # per sq. ft.
Maximum fibre stress is found = 0.551 ton per sq. inch.

e. Apex *E*.

$w' = 10 \cos 38.7°$.

$$w' = 10 \cos 38.7° + \frac{8}{9} \times 38.7 = 42.20 \text{ \# per sq. ft.}$$

$w'' = 10 \sin 38.7° = 6.25$ # per sq. ft.

If purlin be made 3 × 10, maximum fibre stress = 0.579 ton per sq. in., which would be entirely safe. But it is best to make equal

dimensions and equal spacing for purlins throughout, for better appearance.

Maximum fibre stress for $4 \times 10 = 0.482$ ton per sq. inch.

f. Apex F.

$w' = 10 \cos 25.8° + 40 = 31.90$ # per sq. ft. for P and W.

$w' = \cos 25.8° (10 + \cos 25.8° \times 10) = 17.1$ # per sq. ft. P and S.

$w'' = 10 \sin 25.8° = 4.35$ # per sq. ft.

Maximum fibre stress for $4 \times 10 = 0.354$ ton per sq. inch.

g. Apex G.

$$w' = 10 \cos 13° + \frac{8}{9} \times 13 = 22.3 \text{ \# per sq. ft. } P \text{ and } W.$$

$w'' = \cos 13° (10 + 10 \cos 13°) = 19.23$ # per sq. ft. P and S.

$w'' = 10 \sin 13° = 1.74$ # per sq. ft. P and W.

$w'' = \sin 13° (10 + 10 \cos 13°) = 4.44$ # per sq. ft P and S.

Maximum fibre stress for $4 \times 10 = 0.222$ ton per sq. inch.

h. Apex H.

$$w' = 10 \cos 0° + \frac{8}{9} + 0 = 10 \text{ \# per sq. ft. } P \text{ and } W.$$

$w' = \cos 0° (10 + 10 \cos 0) = 20$ # per sq. ft. P and S.

$w'' = 0$ for both P and W and P and S.

It is not necessary to compute maximum fibre stress, since no component W'' exists.

607. Apex Loads on Truss.—The permanent, snow, and wind loads at the apexes are slightly increased from those computed in Chapter IV, since the increased diameter of the surface of the roof increases the areas supported at each apex.

The respective apex loads on the half truss are thus found to be as follows:

	B.	*C*.	*D*.	*E*.	*F*.	*G*.	*H*.
$P =$	1.172	1.172	1.172	1.172	1.172	1.172	1.172 tons.
$S =$	0.200	0.392	0.576	0.712	0.824	0.896	0.464 ton.
$W =$	3.670	3.670	3.670	3.150	2.100	1.060	0.000 ton.

608. Stress Diagrams.—The permanent, snow, and wind stress diagrams are then drawn for the truss diagram given in Fig. 432, similarly to the stress diagrams illustrated in Figs. 99, 100, and 101, allowing for changes in form of truss diagram.

The centre lengths of the members of this truss have already been computed in Example 19 of Chapter VII.

The stresses and lengths are next entered in the stress sheet.

609. Curvature of Members of Upper Chord.—The arc-length of a panel exceeds its chord-length by $\frac{5}{8}''$.

$$\text{The rise of this arc} = 50.0\left(1 - \cos\frac{1}{2}\,12^\circ\,51'\,25\frac{5}{7}''\right) = 0.315 \text{ ft.}$$

$$\text{Arc-length} = 11'\,2\frac{31}{32}'' = 11.2474'.$$

$$\text{Chord-length} = 11'\,2\frac{11}{32}'' = 11.1953'.$$

The radial axes of the purlins divide the arc-length into 4 equal parts.

610. Curvature of Lower Chord.—The arc-length exceeds the chord-length by $\frac{5}{32}''$.

The rise of this arc $= 52.0833\left(1 - \cos\frac{1}{2}\,10^\circ\,32'\,3\frac{2}{7}''\right) =$ 0.222 ft.

$$\text{Arc-length} = 9'\,6\frac{29}{32}'' = 9.5755'.$$

$$\text{Chord-length} = 9'\,6\frac{3}{4}'' = 9.5625'.$$

611. Bending Moment Produced by Maximum Purlin Loads.—Take the member connecting apexes C and D of the upper chord, which supports the maximum purlin loads and moments. The moment diagram is shown in Fig. 586, by which the maximum moment M = 8.5 ft.-tons.

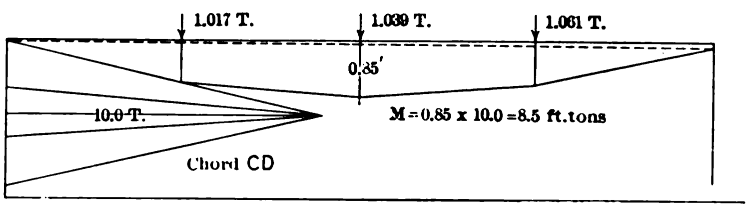


FIG. 586.—Bending Moment on U. C.

612. Dimensioning Upper Chord.—Both legs of angles are riveted. Both straight and curved members will be dimensioned for comparison.

613. A. Members Straight between Apexes.—Member X 1; stresses = − 41.4 and + 5.1 T; $C.L.$ 11′ $2\frac{11}{32}''$.

Apply Table N for compression and G for tension. Try 2, $5 \times 3\frac{1}{2} \times \frac{5}{8}''$ Ls.

Next apply formula 133 for compression and bending moment $M = 8.5$ ft.-tons.

$\frac{Z}{A} + \frac{Mc'}{I} = \frac{41.4}{9.86} + \frac{8.5 \times 1.70}{24.06} = 4.20 + 0.60 = 4.80$ tons per sq. inch.

By Table N, $\frac{48.0}{9.86} = 4.87$ tons per sq. inch = maximum safe compression.

Apply formula 130 for fibre stress in tension.

$\frac{X}{A} + \frac{Mc''}{I} = \frac{5.1}{9.86} + \frac{8.5 \times 3.30}{24.06} = 0.52 + 1.17 = 1.69$ tons per sq. in. tension.

Therefore the proposed section will just suffice to resist either maximum or minimum stresses occurring in it.

Member X 2: stress $= -37.1$ and $+ 5.5$ T; $C.L.$ as before.

By Table N, 2, $5 \times 3\frac{1}{2} \times \frac{9}{16}$ Ls are required for compression alone.

Then $\frac{37.1}{8.94} + \frac{8.5 \times 1.68}{22.06} = 4.15 + .65 = 4.80$ tons per sq. inch compression.

By Table N, $\frac{44.00}{8.94} = 4.93$ tons safe compression per sq. inch.

Also $\frac{5.5}{8.94} + \frac{8.5 \times 3.32}{22.06} = 0.62 + 1.28 = 1.90$ tons per sq. in. tension.

Therefore this section is entirely safe for both kinds of stress.

Member X 4: stress $= -33.8$ and $+ 2.5$ T; $C.L.$ as before.

By Table N, 2, $5 \times 3\frac{1}{2} \times \frac{1}{2}''$ Ls for compression alone.

Then $\frac{33.8}{8.0} + \frac{8.5 \times 1.66}{19.98} = 4.23 + 0.71 = 4.94$ tons per sq. in. compression.

By Table N, $\frac{40.0}{8.0} = 5.00$ tons per sq. inch = maximum safe compression.

And $\dfrac{2.5}{8.0} + \dfrac{8.5 \times 3.34}{19.98} = 0.31 + 1.42 = 1.73$ tons per sq. in. tension.

Member X 6: stress = $-$ 30.7 and $-$ 0.5 T.

Try 2, $5 \times 3\dfrac{1}{2} \times \dfrac{7}{16}''$ Ls.

$\dfrac{30.7}{7.06} + \dfrac{8.5 \times 1.63}{17.80} = 4.35 + 0.78 = 5.13$ tons per sq. in. compression.

By Table N, $\dfrac{35.0}{7.06} = 4.96$ tons per sq. in. = maximum safe compression.

Therefore this section must be increased to 2, $5 \times 3\dfrac{1}{2} \times \dfrac{1}{2}''$ Ls.

Member X 8: stress = $-$ 26.6 and $-$ 3.4 T. Try 2, $5 \times 3\dfrac{1}{2} \times \dfrac{7}{16}''$ Ls.

$\dfrac{26.6}{7.06} + \dfrac{8.5 \times 1.63}{17.80} = 3.77 + 0.78 = 4.55$ tons per sq. in. compression.

By Table N, $\dfrac{35.0}{7.06} = 4.96$ tons per sq. in. = maximum safe compression.

Member X 10: stress = $-$ 22.0 and $-$ 6.27 T. Try 2, $5 \times 3\dfrac{1}{2} \times \dfrac{3}{8}''$ Ls.

$\dfrac{22.0}{6.10} + \dfrac{8.5 \times 1.61}{15.56} = 3.61 + 0.88 = 4.49$ tons per sq. in. compression.

By Table N, $\dfrac{30.5}{6.10} = 5.00$ tons per sq. in. = maximum safe compression.

Member X 12: stress = $-$ 22.6 and $-$ 9.3 T. Try the last section.

$\dfrac{22.6}{6.10} + \dfrac{8.5 \times 1.61}{15.56} = 3.71 + 0.88 = 4.59$ tons per sq. in. compression.

Same as for X 10, 5.00 tons per sq. in. = maximum safe compression.

614. B. Members Curved between Apexes.—Rise of arc = 0.315 ft. Apply formula 137 for compression and 132 for tension.

Member X 1: stress = $-$ 41.4 and $+$ 5.1 T; try 2, $5 \times 3\frac{1}{2} \times 9/16''$ Ls.

$$\frac{41.4}{8.94} + \frac{.315 \times 41.4 \times 1.68 - 8.5 + 1.68}{22.06} = 4.64 + 0.35 = 4.99$$

tons per sq. inch compression.

By Table N, $\frac{44.0}{8.94} = 4.93$ tons per sq. in. safe compression. This will do.

$$\frac{5.1}{8.94} + \frac{.315 \times 5.1 \times 3.32 - 8.5 \times 3.32}{22.06} = 0.57 + 1.04 = 1.61$$

tons per sq. inch tension.

This section is then entirely safe for either kind of stress.

Add the difference between $\frac{X\,a\,c''}{I}$ and $\frac{M\,c''}{I}$ to $\frac{X}{A}$ in all cases.

Member X 2: must have same dimensions as X 1; no splice at apex.

Member X 4: must have same dimensions as X 1; no splice at apex.

Member X 6: stress = $-$ 30.7 and $-$ 0.5 ton; try 2, $5 \times 3\frac{1}{2} \times \frac{7}{16}''$.

First try 2, $5 \times 3\frac{1}{2} \times \frac{3}{8}''$ Ls.

$$\frac{22.6}{6.1} + \frac{.315 \times 22.6 \times 1.61 - 8.5 \times 1.61}{15.56} = 3.81 + 0.14 = 3.94$$

tons per sq. in.

By Table N, $\frac{30.5}{6.1} = 5.00$ tons per sq. in. safe maximum compression.

Hence the section is more than ample, probably 2, $5 \times 3\frac{1}{2} \times \frac{5}{16}''$ would do.

Member X 8: must have same dimensions as X 6; no splice at apex.

Member X 10: must have same dimensions as X 6; no splice at apex.

Member X 12: stress = $-$ 22.6 and $-$ 9.3 T; try 2, $5 \times 3\frac{1}{2} \times \frac{5}{16}''$ Ls.

$$\frac{22.6}{5.12} + \frac{.315 \times 22.6 \times 1.59 - 8.5 \times 1.59}{13.20} = 4.42 + 0.17 = 4.59\ T$$

per sq. in.

615. Lower Chord.—This supports no purlins, but allowance must be made for curvature.

616. A. Members Straight between Apexes.—First apply Table G for tension and Table O for compression.

Member Y 1: stresses = -19.0 and $+ 30.5$ T; $C.L.$ $9'\ 6\frac{3}{4}''$.

By Table G, 2, $5 \times 3\frac{1}{2} \times \frac{3}{8}''$ Ls.

By Table O, 2, $4 \times 3 \times \frac{5}{16}''$ Ls.

Member Y 3: stress = $-$ 8.5 and $+$ 28.4 T; $C.L.$ as before.

By Table G, 2, $5 \times 3\frac{1}{2} \times \frac{3}{8}''$ Ls.

By Table O, 2, $4 \times 3 \times \frac{5}{16}''$ Ls.

Member Y 5: stress = $-$ 9.9 and $+$ 25.7 T; $C.L.$ as before.

By Table G, 2, $5 \times 3\frac{1}{2} \times \frac{5}{16}''$ Ls.

By Table O, 2, $4 \times 3 \times \frac{5}{16}''$ Ls.

Member Y 7: stress = $-$ 5.7 and $+$ 21.8$''$; $C.L$ as before.

By Table G, 2, $5 \times 3 \times \frac{5}{16}''$ Ls.

By Table O, 2, $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}''$ Ls.

Member Y 9: stress = $-$ 1.6 and + 19.1 T; $C.L.$ as before.

By Table G, 2, $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}''$ Ls.

By Table O, 2, $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}''$ Ls.

Member Y 11: stress = + 2.3 and + 20.3 T; $C.L.$ as before.

By Table G, 2, $3\frac{1}{2} \times 3 \times \frac{5}{16}''$ Ls.

Member Y 13: stress = + 6.8 and + 20.8 T; $C.L.$ as before.

By Table G, 2, $3\frac{1}{2} \times 3 \times \frac{5}{16}''$ Ls.

617. B. Members Curved Between Apexes.—Rise of arc = .222 ft. for $9'\ 6\frac{3}{4}''$.

Apply formula 135 for compressile stress.

Apply formula 130 for tensile stress. In the last case only must a proper deduction be made for all rivet holes in a cross-section of member.

Member Y 1: stress = $-$ 19.0 and + 30.5 T; $C.L.$ $9'\ 6\frac{3}{4}''$.

Try 2, $5 \times 3\frac{1}{2} \times \frac{3}{8}''$ Ls.

$$\frac{Z}{A} + \frac{Z\,a\,c'}{I} = \frac{19.0}{6.10} + \frac{.222 \times 19.9 \times 1.61}{15.56} = 3.12 + 0.44 = 3.56 \quad T$$

per sq. in.

$$\frac{X}{A} + \frac{X\,a\,c''}{I} = \frac{30.5}{6.1 - 1.96} + \frac{.222 \times 30.5 \times 3.39}{15.56} = 7.37 + 1.47 =$$

8.84 T per sq. in.

Try 2, $5 \times 3\frac{1}{2} \times \frac{7}{16}''$ Ls. Apply formula 130 only.

$$\frac{30.5}{7.06 - 2.30} + \frac{.222 \times 30.5 \times 3.37}{17.80} = 6.41 + 1.28 = 7.69 \ T \text{ per}$$

sq. in.

Member Y 3: must have same dimensions as Y 1; no splice at apex.

Member Y 5: must have same dimensions as Y 1; no splice at apex.

Member Y 7: stress = $-$ 5.7 and + 21.8 T; $C.L.$ as before.

Try 2, $5 \times 3\frac{1}{2} \times \frac{5}{16}''$ Ls.

$$\frac{5.70}{5.12} + \frac{.222 \times 5.7 \times 1.59}{13.20} = 1.11 + 0.33 = 1.44\ T \text{ per sq. inch.}$$

$$\frac{21.8}{5.12 - 1.64} + \frac{.222 \times 21.8 \times 3.41}{13.20} = 6.27 + 1.25 = 7.52\ T \text{ per sq. inch.}$$

Member Y 9: must have same dimensions as Y 7; no splice at apex.

Member Y 11: stress = + 2.3 and + 20.3 T; C. L. as before.

Try 2, $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{8}''$ Ls.

$$\frac{20.3}{4.22 - 1.32} + \frac{.222 \times 20.3 \times 2.34}{5.12} = 7.01 + 0.20 = 7.21\ T \text{ per sq. inch.}$$

Member Y 13: make same dimensions as Y 7.

618. Web Radials.—

Member 1 2: stress = $-$ 1.6 and + 5.9 T; $C.L.$ $3'\ 0\frac{1}{8}''$.

By Table M, 2, $2 \times 2 \times \frac{3}{16}''$ Ls.

By Table G, 2, $2 \times 2 \times \frac{3}{16}''$ Ls. B.F.

Member 3 4: stress = 0.0 and + 4.1 T; $C.L.$ $5'\ 11\frac{1}{4}''$.

By Table G, 2, $2 \times 2 \times \frac{3}{16}''$ Ls.

Member 5 6: stress = + 1.8 and + 5.8 T; $C.L.$ $7'\ 11\frac{1}{2}''$.

By Table G, 2, $2 \times 2 \times \frac{3}{16}''$ Ls.

Member 7 8: stress = $-$ 0.2 and + 6.0 T; $C.L.$ $10'\ 1\frac{1}{16}''$.

By Table O, 2, $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}''$ Ls.

By Table G, 2, $2 \times 2 \times \frac{3}{16}''$ Ls.

Member 9 10: stress = -1.8 and $+5.4$ T; *C.L.* 11′ $5\frac{1}{16}$″.

By Table O, 2, $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$″ Ls.

By Table G, 2, $2 \times 2 \times \frac{3}{16}$″ Ls.

Member 11 12: stress = -3.4 and $+5.1$ T; *C.L.* 12′ $2\frac{3}{4}$″.

By Table O, 2, $4 \times 3 \times \frac{5}{16}$″ Ls.

By Table G, 2, $2 \times 2 \times \frac{3}{16}$″ Ls.

Member 13 13′: stress = $+2.3$ and $+3.8$ T; *C.L.* 12′ 6″.

By Table G, 2, $2 \times 2 \times \frac{3}{16}$′ Ls.

619. Web Diagonals.—

Member 2 3: stress = -9.7 and -3.7 T; *C.L.* 12′ $10\frac{13}{16}$″.

By Table O, 2, $4 \times 3 \times \frac{5}{16}$″ Ls.

Member 4 5: stress = -9.4 and -3.5 T; *C.L.* 14′ $4\frac{3}{8}$″.

By Table N, 2, $5 \times 3\frac{1}{2} \times \frac{5}{16}$″ Ls.

Member 6 7: stress = -8.6 and -3.0 T; *C.L.* 15′ $2\frac{3}{8}$″.

By Table N, 2, $5 \times 3\frac{1}{2} \times \frac{5}{16}$″.

Member 8 9: stress = -8.0 and -2.2 T; *C.L.* 16′ $3\frac{5}{32}$″.

By Table N, 2, $6 \times 4 \times \frac{3}{8}$″ Ls.

Member 10 11: stress = -7.5 and -1.4 T; *C.L.* 16′ $6\frac{5}{32}$″.

By Table N, 2, $6 \times 4 \times \frac{3}{8}$″ Ls.

Member 12 13: stress = − 7.0 and − 0.5 T; $C.L.$ 16′ 5″.

$$\text{By Table N, 2, } 6 \times 4 \times \frac{3}{8}\text{″ Ls.}$$

A series of different types of roofs and their trusses have been fully dimensioned in this chapter, in order to explain clearly the methods employed and the application of the formulas and tables given in earlier chapters.

The stress sheet for this example is given in Art. 620.

The results of dimensioning members for Cases A and B are to be found in the dimension sheet, Art. 621.

620. Stress Sheet for Examples 10 A and 10 B.—

Member.	P-stress.	S-stress W.	W-stress W.	L-stress.	− Maximum.	+ Maximum.	C-length.
X 1	−10.8	− 6.2	−30.6	+15.9	−41.4	+ 5.1	11′ 2 11/32″
X 2	− 8.4	− 5.1	−28.7	+13.9	−37.1	+ 5.5	11′ 2 11/32″
X 4	− 9.4	− 5.9	−24.4	+11.9	−33.8	+ 2.5	11′ 2 11/32″
X 6	−10.3	− 6.8	−20.4	+ 9.8	−30.7	− 0.5	11′ 2 11/32″
X 8	−11.0	− 7.4	−15.6	+ 7.6	−26.6	− 3.4	11′ 2 11/32″
X 10	−11.5	− 8.0	−10.5	+ 5.3	−22.0	− 6.2	11′ 2 11/32″
X 12	−11.8	− 8.4	−10.8	+ 2.5	−22.6	− 9.3	11′ 2 11/32″
Y 1	+ 3.3	+ 1.9	+27.2	−22.3	−19.0	+30.5	9′ 6¾″
Y 3	+ 6.0	+ 3.7	+22.4	−14.5	− 8.5	+28.4	9′ 6¾″
Y 5	+ 8.3	+ 5.3	+17.4	−18.2	− 9.9	+25.7	9′ 6¾″
Y 7	+10.1	+ 6.8	+11.7	−15.8	− 5.7	+21.8	9′ 6¾″
Y 9	+11.3	+ 7.8	+ 5.7	−12.9	− 1.6	+19.1	9′ 6¾″
Y 11	+11.9	+ 8.4	+ 5.2	− 9.6	+ 2.3	+20.3	9′ 6¾″
Y 13	+12.1	+ 8.7	− 3.3	− 5.3	+ 6.8	+20.8	9′ 6¾″
1 2	+ 2.3	+ 1.4	+ 3.6	− 3.9	− 1.6	+ 5.9	3′ 0⅛″
3 4	+ 3.0	+ 2.1	+ 1.1	− 3.0	− 0.0	+ 4.1	5′ 11¼″
5 6	+ 3.4	+ 2.4	− 1.3	− 1.6	+ 1.8	+ 5.8	7′ 11½″
7 8	+ 3.4	+ 2.6	− 3.6	− 0.5	− 0.2	+ 6.0	10′ 1 1/16″
9 10	+ 3.1	+ 2.3	− 4.9	+ 1.0	− 1.8	+ 5.4	11′ 5 1/16″
11 12	+ 2.5	+ 1.8	− 5.9	+ 2.6	− 3.4	+ 5.1	12′ 2¾″
13 13′	+ 2.3	+ 1.5	+ 1.0	+ 1.0	+ 2.3	+ 3.8	12′ 6″
2 3	− 3.7	− 2.3	− 0.6	− 6.0	− 9.7	− 3.7	12′ 10 13/16″
4 5	− 3.5	− 2.4	− 1.6	− 5.9	− 9.4	− 3.5	14′ 4⅜″
6 7	− 3.0	− 2.3	− 2.5	− 5.6	− 8.6	− 3.0	15′ 2⅜″
8 9	− 2.2	− 2.0	− 3.6	− 5.8	− 8.0	− 2.2	16′ 3 5/32″
10 11	− 1.4	− 1.2	− 4.7	− 6.1	− 7.5	− 1.4	16′ 6 5/32″
12 13	− 0.5	− 0.5	− 6.0	− 6.5	− 7.0	− 0.5	16′ 5″

621. Dimension Sheet for Examples 10 A and 10 B.—

Member.	Members Straight.		Members Curved.	
	Dimensions	S. S. rivets.	Dimensions.	S. S. rivets.
X 1..........	2, $5\times3\frac{1}{2}\times\frac{5}{8}''$ Ls.	16	2, $5\times3\frac{1}{2}\times\frac{9}{16}''$ Ls.	16
X 2..........	2, $5\times3\frac{1}{2}\times\frac{9}{16}''$ Ls.	14	2, $5\times3\frac{1}{2}\times\frac{9}{16}''$ Ls.	14
X 4..........	2, $5\times3\frac{1}{2}\times\frac{1}{2}''$ Ls.	13	2, $5\times3\frac{1}{2}\times\frac{9}{16}''$ Ls.	13
X 6..........	2, $5\times3\frac{1}{2}\times\frac{1}{2}''$ Ls.	12	2, $5\times3\frac{1}{2}\times\frac{7}{16}''$ Ls.	12
X 8..........	2, $5\times3\frac{1}{2}\times\frac{7}{16}''$ Ls.	11	2, $5\times3\frac{1}{2}\times\frac{7}{16}''$ Ls.	11
X 10.........	2, $5\times3\frac{1}{2}\times\frac{3}{8}''$ Ls.	9	2, $5\times3\frac{1}{2}\times\frac{7}{16}''$ Ls.	9
X 12.........	2, $5\times3\frac{1}{2}\times\frac{3}{8}''$ Ls.	9	2, $5\times3\frac{1}{2}\times\frac{3}{8}''$ Ls.	9
Y 1..........	2, $5\times3\frac{1}{2}\times\frac{3}{8}''$ Ls.	12	2, $5\times3\frac{1}{2}\times\frac{7}{16}''$ Ls.	12
Y 3..........	2, $5\times3\frac{1}{2}\times\frac{3}{8}''$ Ls.	11	2, $5\times3\frac{1}{2}\times\frac{7}{16}''$ Ls.	11
Y 5..........	2, $5\times3\frac{1}{2}\times\frac{5}{16}'$, Ls.	10	2, $5\times3\frac{1}{2}\times\frac{7}{16}''$ Ls.	10
Y 7..........	2, $5\times3\frac{1}{2}\times\frac{5}{16}''$ Ls.	9	2, $5\times3\frac{1}{2}\times\frac{5}{16}''$ Ls.	9
Y 9..........	2, $3\frac{1}{2}\times2\frac{1}{2}\times\frac{5}{16}''$ Ls.	8	2, $5\times3\frac{1}{2}\times\frac{5}{16}''$ Ls.	8
Y 11.........	2, $3\frac{1}{2}\times2\frac{1}{2}\times\frac{5}{16}''$ Ls.	8	2, $5\times3\frac{1}{2}\times\frac{5}{16}''$ Ls.	8
Y 13.........	2, $3\frac{1}{2}\times2\frac{1}{2}\times\frac{5}{16}''$ Ls.	8	2, $5\times3\frac{1}{2}\times\frac{5}{16}''$ Ls.	8
1 2..........	2, $2\times2\times\frac{3}{16}''$ Ls.	4	2, $2\times2\times\frac{3}{16}''$ Ls.	4
3 4..........	2, $2\times2\times\frac{3}{16}''$ Ls.	3	2, $2\times2\times\frac{3}{16}''$ Ls.	3
5 6..........	2, $2\frac{1}{2}\times2\times\frac{3}{16}''$ Ls.	4	2, $2\frac{1}{2}\times2\times\frac{3}{16}''$ Ls.	4
7 8..........	2, $3\frac{1}{2}\times2\frac{1}{2}\times\frac{1}{4}''$ Ls.	4	2, $3\frac{1}{2}\times2\frac{1}{2}\times\frac{1}{4}''$ Ls.	4
9 10.........	2, $3\frac{1}{2}\times2\frac{1}{2}\times\frac{1}{4}''$ Ls.	4	2, $3\frac{1}{2}\times2\frac{1}{2}\times\frac{1}{4}''$ Ls.	4
11 12.........	2, $4\times3\times\frac{5}{16}''$ Ls.	2	2, $4\times3\times\frac{5}{16}''$ Ls.	2
13 13′........	2, $2\times2\times\frac{3}{16}''$ Ls.	3	2, $2\times2\times\frac{3}{16}''$ Ls.	3
2 3..........	2, $4\times3\times\frac{5}{16}''$ Ls.	4	2, $4\times3\times\frac{5}{16}''$ Ls.	4
4 5..........	2, $5\times3\frac{1}{2}\times\frac{5}{16}''$ Ls.	4	2, $5\times3\frac{1}{2}\times\frac{5}{16}''$ Ls.	4
6 7..........	2, $5\times3\frac{1}{2}\times\frac{5}{16}''$ Ls.	4	2, $5\times3\frac{1}{2}\times\frac{5}{16}''$ Ls.	4
8 9..........	2, $6\times4\times\frac{3}{8}''$ Ls.	4	2, $6\times4\times\frac{3}{8}''$ Ls.	4
10 11.........	2, $6\times4\times\frac{3}{8}''$ Ls.	3	2, $6\times4\times\frac{3}{8}''$ Ls.	3
12 13.........	2, $6\times4\times\frac{3}{8}''$ Ls.	3	2, $6\times4\times\frac{3}{8}''$ Ls.	3

CHAPTER XII

DETAILING CONNECTIONS AT APEXES OF ROOFS

622. General Remarks.—In order to preserve the relations of these connections at apexes to each other and to clearly exhibit their construction, it is most convenient to employ two different scales in the illustrations of this chapter and in practice; one for the centre lines of the half-truss diagram, larger than that used in Chapter IV and comprising centre lines of all members; also a much larger scale for the details of the connections themselves. Both side and top or end views are sometimes required to make clear an end connection. From such a drawing at proper scales may readily be taken all data required for estimates or bills of material, framing bevels, templates, or full-size details of connections, if these are necessary.

As a general rule, connections must be as simple as possible, to economize materials and labor in construction and erection. All stresses acting at an apex are to be considered and provision for resisting them safely be made, excepting that the small moments usually occurring in riveted connections may usually be neglected.

As illustrations of the methods of making details of connections, the examples dimensioned in Chapter XI will be successively detailed in this chapter.

2.—Example 1.—A Wooden Truss with Extra Loads

The connections of the members of this truss are detailed here in accordance with the dimensions found in Example 1 of the last chapter.

623. Sheathing.—Composed of $\frac{7}{8}''$ matched pine through nailed on the rafters (Fig. 588).

624. Rafters.—These are $2 \times 6''$, full size, set $27''$ on centres, with ends boxed to 5 deep, then butted and spiked to purlins as in Fig. 588. Or the ends are often lapped sidewise on purlin.

625. Purlins.—These are frequently halved down on top of the principal, or their ends are housed into it $2''$ on each side. But a better method is shown in Fig. 589, where the ends of the purlins

are butted against side of principal, then being held in place by 2, 1 ft. long, $3 \times 3 \times \frac{1}{4}''$ Ls, fastened to purlin and principal by 4″ lagscrews. This looks better, since purlins and principal are framed flush on top and bottom, and the upper nuts on rods remain accessible.

626. Ceiling Joists.—These are also 2 x 6″, full size, though the ordinary size would suffice. Fig. 590 is a transverse section across the ceiling. They support a protecting floor not intended for loads and the lath and plaster ceiling. Their ends are usually supported by strips spiked to sides of lower chord of truss.

627. Upper Chord.—To prevent slipping of principals at ridge apex, it is well to cut a tenon on one fitting a recess cut in the other, as shown in Fig. 587. It is simplest to scarf the lower end of principal to fit on top of lower chord at its end, without indents, keys, hoops, or bolts, depending entirely on steel fish-plates at sides and through bolts. More complicated connections are generally employed at an increased cost and possess uncertain resistance to stresses.

At least one splice must be made here in each principal, which should be placed just above an apex. Since the member is subject to longitudinal compression, if the splice be correctly cut, the stress will be directly transmitted by contact of the ends of the timbers. Hence 2 or 4 bolts will be sufficient to hold together the splice (Fig. 587).

628. Lower Chord.—Besides the connections at the ends of the truss, splices are here made at the middle of $Y\,4$ and at the centre of the span, so as not to require timbers of unusual length and cost. Bolts 1″ in diameter are here used.

Apply formulas for spliced timbers in tension, Art. 482, Chapter IX.

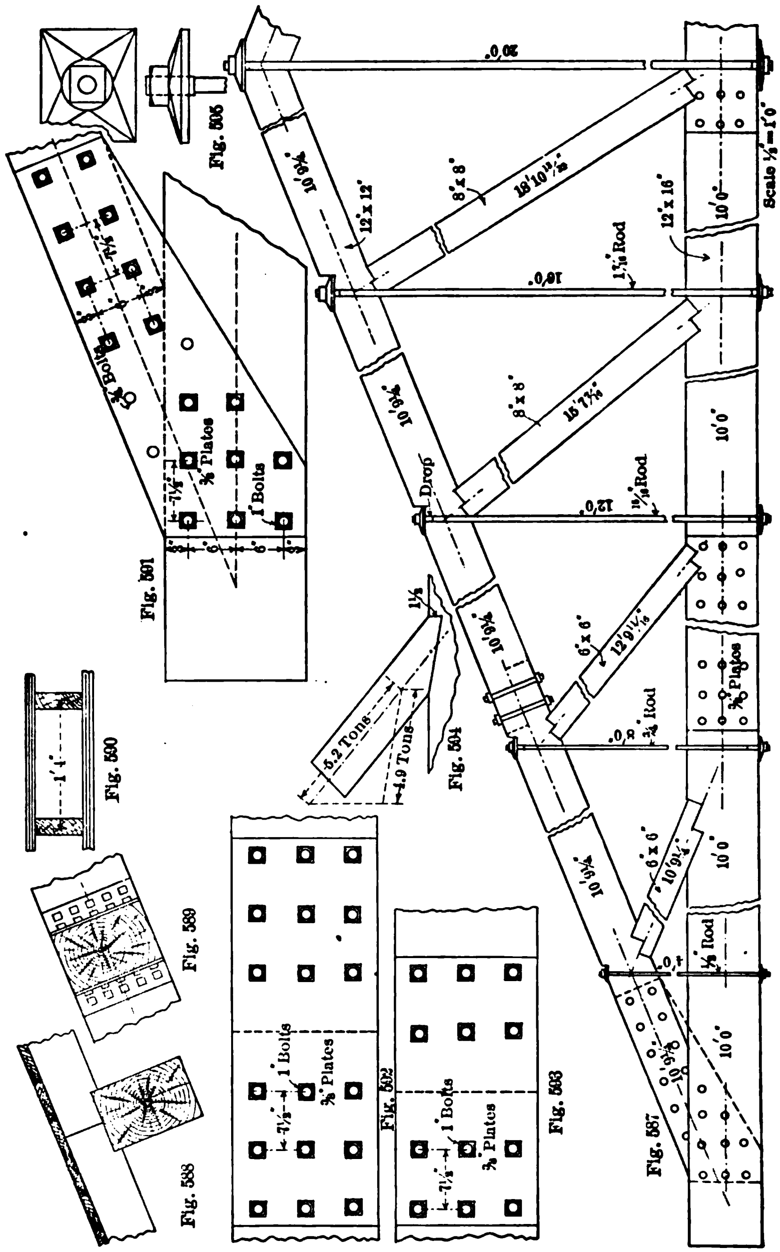
$g = \frac{d\,C}{2\,S'} = \frac{1 \times 0.60}{2 \times 0.04} = 7.5$ ins. between centres of bolts and to end of timber.

$t = \frac{3}{8} \times d = \frac{3}{8}$ in. = minimum thickness of fish-plates.

629. Splices.—Connection of $X\,1$ and $Y\,1$ at ends of truss (Fig. 591).

$n = 2$ rows of 1″ bolts in upper chord, 3 rows of same in lower chord.

$$t = \frac{X}{16\,(h - n\,d)} = \frac{16\,(12 - 2 \times 1)}{55.9} = \frac{55.9}{160} = 0.35'', \text{ say } \frac{3}{8}'' \text{ fish-plates.}$$



Details of Wooden Truss. Example 1

$N = \frac{X}{b\,d\,C} = \frac{55.9}{12 \times 1 \times 0.60} - \frac{55.9}{7.20} = 7.76$, say 8 bolts in upper chord.

$N = \frac{X}{b\,d\,C} = \frac{51.1}{12 \times 1 \times 0.60} - \frac{51.1}{7.20} = 7.10$, say 8 bolts in lower chord.

This connection is indicated in Fig. 587 and is detailed at a larger scale in Fig. 591.

Splice in $Y\,4$ (Fig. 587).

$t = \frac{X}{16\,(h - n\,d)} = \frac{46.4}{16\,(18 - 3 \times 1)} = \frac{46.4}{108} = 0.223''$, say $\frac{1}{4}''$ fish-plates. But these must be made at least $\frac{3}{8}''$ thick to develop full shear in bolts.

$N = \frac{X}{b\,d\,C} = \frac{46.4}{12 \times 1 \times 0.60} - \frac{46.4}{7.20} = 6.44$, say 7 bolts in each end.

Splice at middle of truss (Figs. 587 and 593).

$t = \frac{36.7}{208} = 0.177''$, say $\frac{3}{16}''$. But this must be made $\frac{3}{8}''$ as before.

$N = \frac{36.7}{7.20} = 5.1$, say 6 bolts in each end of splice.

630. Web Struts.—These are square timbers, and their ends are most simply halved and boxed into the upper and lower chords as in Fig. 587, at right angles to axis of the strut. These may be merely set in place, be spiked in, have stub tenons on both ends, or be fixed by two dowels of steel or wood in each end.

Splayed boxings are common, but are more difficult to fit and have no advantages. They are more affected by shrinkage of timbers (Fig. 594).

The indents may be conveniently laid off on each side of the chord, then sawn across and completed with chisel and rebate plane.

631. Web Ties.—The ends of these vertical steel rods are best enlarged by upsetting them in standard steel dies, so as to make each end stronger than the rod itself, after cutting the screws. It is more economical to use them, if they can be obtained from shops fitted with dies and machines. But on no account must the upset ends be welded on the rods. If it be not entirely certain that rods with properly upset ends are to be used, then use Table A for rods without upset ends, in determining dimensions of rods.

Drop = length of upset end measured from face of washer.

Member 1 2: maximum stress = 0.0 ton.

One $\frac{1}{2}''$ rod without upsets should be used, since it supports the weight of a panel of the lower chord. By Table A, nut is $\frac{1}{2} \times \frac{3}{4}''$. For shortleaf pine, $C' = 0.125$. Hence net area of washer = 8.1 sq. ins. Hole through washer is always made $\frac{1}{16}''$ larger than diameter of rod end, or $\frac{9}{16}''$, and the corresponding area = 0.25 sq. in. Hence total area of washer = 8.1 + 0.25 = 8.35 sq. ins., and $\sqrt{8.35} = 2.88$ ins. square. Its thickness always = diameter of rod end, here $\frac{1}{2}$ inch, and the washer is tapered to $\frac{1}{6}''$, or $\frac{1}{3}$ its thickness, at its edge.

Dimensions are computed in the same manner for the remaining rods, etc.

Member 3 4: maximum stress = 3.2 tons.

By Table B: 1, $\frac{3}{4}''$ rod; upset end, $1 \times 4\frac{1}{2}''$; nut, $1 \times 1\frac{3}{4}''$; drop, 2″; washer, $5\frac{3}{8}''$ square, $\frac{3}{4}''$ under nut, $\frac{1}{4}''$ at edge.

Member 5 6: maximum stress = 5.0 tons.

By Table B: $\frac{15}{16}''$ rod; upset ends, $1\frac{1}{4} \times 4\frac{3}{4}''$; nut, $1\frac{1}{4} \times 2\frac{1}{4}''$; drop, $1\frac{3}{4}''$; washer, $6\frac{3}{4}''$ square, $1\frac{1}{4}''$ under nut, $\frac{3}{8}''$ at edge.

Member 7 8: maximum stress = 12.0 tons.

By Table B: 1, $1\frac{7}{16}''$ rod; upsets, $1\frac{7}{8}'' \times 5\frac{1}{2}''$; nut, $1\frac{7}{8}'' \times 3\frac{3}{4}''$; drop, $1\frac{1}{4}''$; washer, $10\frac{5}{16}''$ square or $8\frac{7}{8}'' \times 12''$, $1\frac{7}{8}''$ under nut, $\frac{5}{8}''$ at edge.

Member 9 9′: maximum stress = 20.6 tons (Fig. 595).

By Table B: 1, $1\frac{13}{16}''$ rod; upsets, $2\frac{1}{4}'' \times 5\frac{3}{4}''$; nut, $2\frac{1}{4}'' \times 4\frac{1}{4}''$; drop, $\frac{3}{4}''$; washer, $14'' \times 12''$, $2\frac{1}{4}''$ under nut, $\frac{3}{4}''$ at edge.

The drops under washers should be indicated on the detail drawings as in Fig. 587, since this is required in estimating the weights of the rods, ends, nuts, and washers, as explained in Chapter XIV.

632. Splayed Boxings.—If splayed boxings are preferred at the ends of web struts, the depth at toe is assumed and then checked as indicated in Fig. 594, for the strut 4 5 with a maximum stress of − 6.2 tons. The depth at toe is here taken at $1\frac{1}{2}''$ and the boxing is then drawn. The stress is next laid off on axis of strut and resolved into components parallel to the lines of the boxing. Then the compression at toe = 4.9 tons, and $\frac{4.9}{1.5 \times 6 \times 0.6} = 0.908''$ deep. Hence this is amply safe and it might be reduced in depth by repeating the process.

633. Example 2. A Wooden Truss.—Resume Example 2 of Chapters IV and XI. The same tables and formulas are then applied as in Example 1. The dimension sheet in Chapter XI shows that the stresses and dimensions of the members are all somewhat less than in Example 1, making a lighter truss, to be compared with that of Example 3 in steel, supporting nearly the same loads. The half elevation of the detail diagram is shown in Fig. 596. Dimensions for all members are obtained in the same manner as for Example 1.

634. Splices.—Connection of $X\,1$ and $Y\,1$ (Fig. 598).

n = 2 rows of 1″ bolts in upper chord.

n = 3 rows of 1″ bolts in lower chord.

$t = \frac{3}{8} \times d = \frac{3}{8}''$ = minimum thickness of fish-plates.

$g = \frac{d\,C}{2\,S'} = \frac{1 \times 0.6}{2 \times .04} = 7.5''$ between centres of bolts or to end of timber.

$t = \frac{X}{16\,(h - n\,d)} = \frac{33.2}{16\,(12 - 2 \times 1)} = \frac{33.2}{160} = .208$, say $\frac{7}{32}''$ plates for upper chord. But these plates must be at least $\frac{3}{8}''$ thick at all connections.

$N = \frac{X}{b\,d\,C} = \frac{33.2}{10 \times 1 \times 0.6} = \frac{33.2}{160} = 5.54$, say 6 bolts in upper chord.

$N = \dfrac{X}{b\,d\,C} = \dfrac{29.8}{10 \times 1 \times .60} = \dfrac{29.8}{160} = 4.97$, say 5 bolts in lower chord.

Splice in Y 4 (Fig. 599).

$t = \dfrac{X}{16\,(h - n\,d)} = \dfrac{26.6}{16\,(12 - 2 \times 1)} = \dfrac{26.6}{160} = .166$, say $\dfrac{3}{16}''$. To be $\dfrac{3}{8}''$.

$N = \dfrac{X}{b\,d\,C} = \dfrac{26.6}{10 \times 1 \times 0.6} = \dfrac{26.6}{6.00} = 4.7$, say 5 bolts in each end.

Splice at middle of span (Fig. 600).

$t = \dfrac{X}{16\,(h - n\,d)} = \dfrac{20.0}{16\,(12 - 2 \times 1)} = \dfrac{20.0}{160} = .156$, say $\dfrac{3}{16}''$ plates. To be $\dfrac{3}{8}''$.

$N = \dfrac{X}{b\,d\,C} = \dfrac{20.0}{10 \times 1 \times 0.6} = \dfrac{20.0}{160} = 3.3$, say 4 bolts in each end.

635. Web Ties.—Steel rods with upset ends, except for 1 2; dimensions found as in last example.

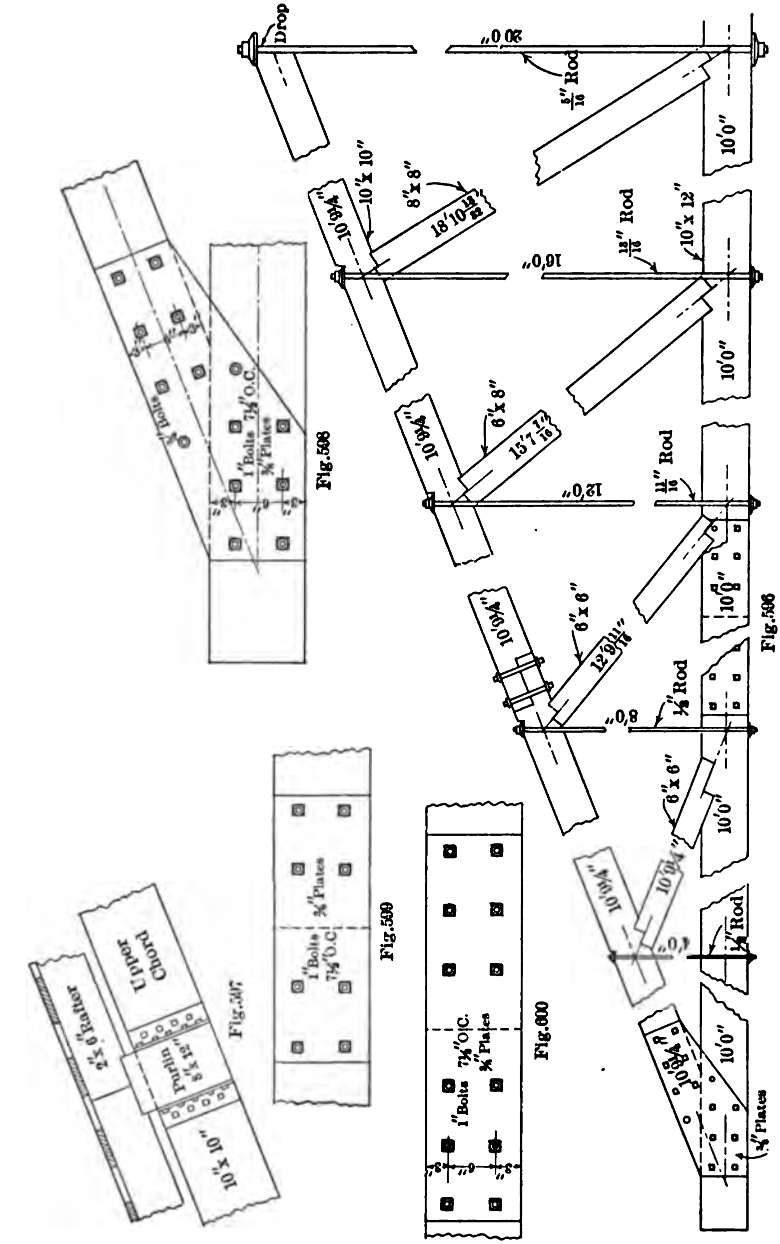
Member 1 2: $\dfrac{1}{2}''$ rod with ends not upset; nuts $\dfrac{1}{2} \times \dfrac{3}{4}''$; washer $2\dfrac{7}{8}''$ square, $\dfrac{1}{2}''$ thick under nut, $\dfrac{1}{6}''$ at edge.

Member 3 4: $1\dfrac{5}{8}''$ rod; upset end, $\dfrac{7}{8} \times 4\dfrac{1}{2}''$; nut $\dfrac{7}{8}'' \times 1\dfrac{1}{2}''$ square; drop $2\dfrac{1}{4}''$; washer, $4\dfrac{1}{2}''$ square, $\dfrac{7}{8}''$ under nut, $\dfrac{5}{16}''$ at edge.

Member 5 6: $\dfrac{13}{16}''$ rod; upset end, $1\dfrac{1}{8}'' \times 4\dfrac{3}{4}''$; nut, $1\dfrac{1}{8} \times 2''$; drop $2''$; washer $5\dfrac{13}{16}''$ square; $1\dfrac{1}{8}''$ under nut, $\dfrac{3}{8}''$ edge.

Member 7 8: $1''$ rod; upset end, $1\dfrac{3}{8} \times 5''$; nut, $1\dfrac{3}{8} \times 2\dfrac{3}{4}''$ square; drop, $1\dfrac{3}{4}''$; washer, $7\dfrac{3}{16}''$ square; $1\dfrac{3}{8}''$ under nut, $\dfrac{7}{16}''$ edge.

Member 9 9′: $1\dfrac{1}{2}''$ rod; upset end, $2 \times 5\dfrac{1}{2}''$; nut $2 \times 4''$; drop, $1''$; washer, $10 \times 10\dfrac{3}{4}''$, $2''$ under nut, $\dfrac{5}{8}''$ edge.



Details of Wooden Truss. Example 2

636. Detail Drawings.—Half elevation of detail diagram is shown in Fig. 596; connection of sheathing, rafters, purlin, and principal in Fig. 597; connections of X 1 and Y 1 in Fig. 598; splice in Y 4 in Fig. 599, and that at middle of span in Fig. 600.

Example 3.—A Steel Truss for the Same Roof

637. General Remarks.—In detailing a steel truss composed of angles, the rivet line on the wider leg and nearest the back of the angle should be set to coincide with the centre line of the member in the truss diagram, when both legs are riveted. This is best in all cases, but when only the wider leg, parallel to the middle vertical plane of the truss, is riveted, and this wider leg has two rows of rivets, the rivet lines are sometimes set equidistant from the centre line of the member. A single row of rivets in the wider leg should always be set on the centre line of the member. This is usual and it is more convenient in design and construction in the shops, than to attempt to locate the centre of gravity of the section of the member on the centre line on the truss diagram, as is sometimes done in books.

Use $\frac{3}{4}''$ rivets for trusses of 100 ft. span or less; $\frac{7}{8}''$ for wider spans or heavy framing.

Gusset plates are to be of uniform thickness throughout the truss, not over $\frac{1}{8}''$ less in thickness than the diameter of the rivet used, so as to develop the full resistance of the rivet to shearing.

It is always advisable and more economical to rivet both legs of each angle, and not less than 2 rivets should be placed in each connection.

Chain riveting is best in angles of sufficient width of leg; staggered riveting otherwise and also in different legs, in order to use the riveting machine, which requires $\frac{3}{8}''$ clearance.

Intermediate riveted connections are to be inserted between angles composing a member in compression, at proper distances. These angles are often bent apart considerably, but this is seldom necessary, since the member is weakest usually in the vertical plane of the truss and not sidewise.

Pitch of rivets in trusses to be 3 diameters at connections, with 2 diameters' distance from centre of rivet to end of piece.

638. Sheathing, Rafters, and Purlins (Figs. 604, 605).—Fig. 604 shows the mode of fastening a purlin on the upper chord at a splice therein. The purlin is boxed down $\frac{5}{16}''$ = thickness of top cover plate at the splice, when the rivets are sunk in holes in the purlin. The purlin is then held by lagscrews at bottom and one side, passing through a bracket composed of a steel plate and angles on each side, the bracket being held to upper chord by rivets or bolts. Or the bracket might be of cast iron if preferred, bolted to upper chord.

Fig. 605 shows a purlin attached directly to upper chord and not boxed, having its top edge in the same plane as the former.

639. Rivets, Gussets, and Covers.—Table T gives the number of single-shear rivets required at each end of each member of the truss and at each splice. Half this number at least appear on the elevation of the detail diagram, and these are divided between the legs of the angles in proportion to the number of rivet lines in each leg.

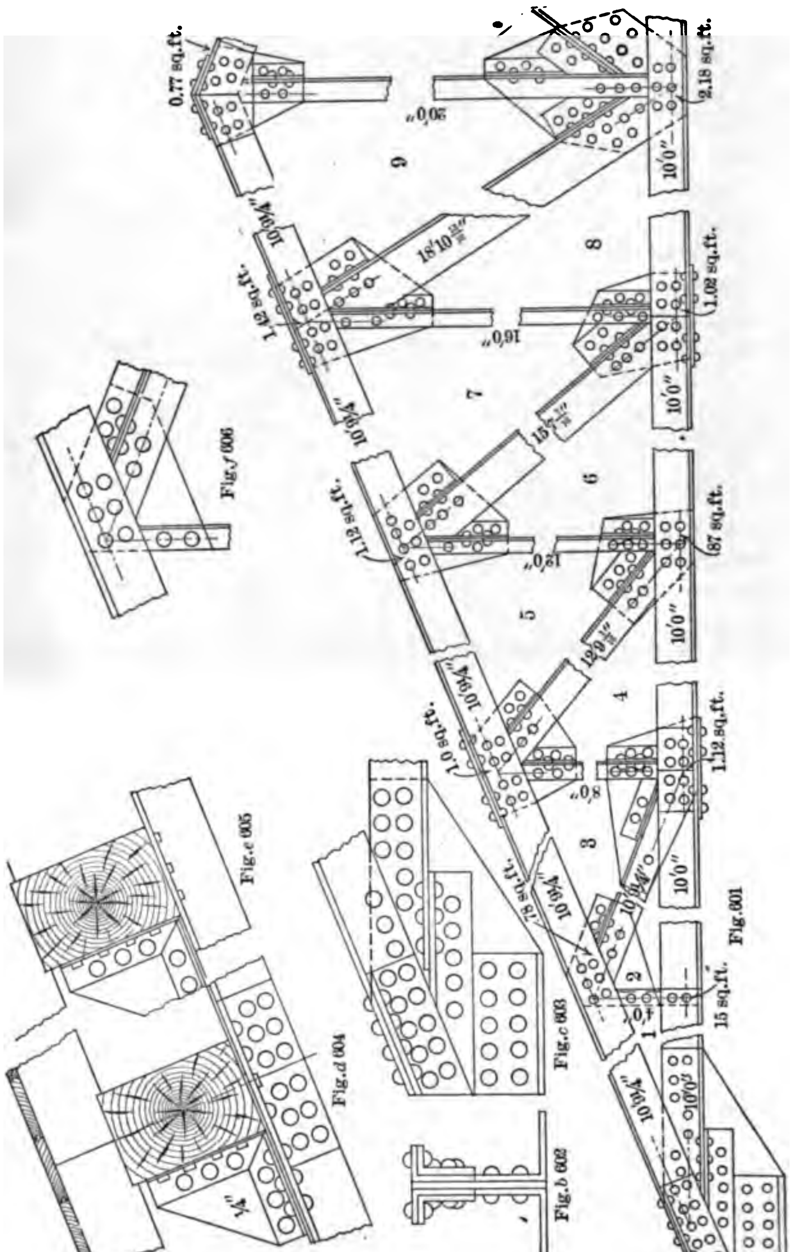
Gusset plates are all $\frac{5}{8}''$ thick for $\frac{3}{4}''$ rivets to develop full shear in rivet; cover plates are $\frac{5}{16}''$ at splices; rivets are set $1\frac{1}{2}''$ from ends and $2\frac{1}{4}''$ between centres, staggered in opposite legs.

Rivets are to be arranged symmetrically about the centre line of the member where possible, but some extra rivets are necessarily inserted at end connections, as in Fig. 603.

The maximum stress in the member 1 2 being 0.0 ton, it is not necessary to connect more than one leg of each angle, but this is done in all other cases.

640. Connection at End of Truss.—Fig. 602 is an end, and Fig. 603 a side elevation of the connection at end of truss, enlarged at base to rest on the masonry wall. The parallel or web legs of the upper chord are held by rivets through the gusset; its horizontal legs are then riveted to short angles milled or forged to fit closely between the web and top legs, and these angles are then riveted through the web legs and the gusset.

The lower chord has its web legs riveted through the gusset, the bottom legs being riveted to short angles, that are in turn riveted through the gusset.



Details of Steel Truss. Example 3

Two $6 \times 6 \times \frac{1}{2}''$ short angles are next riveted through the gusset to form a broad base to rest on the màsonry of the wall. These should be held in place by anchor bolts extending down into the wall.

The upper and lower chords are here spliced at alternate apexes, the entire stress at each splice being transmitted through the rivets, the cover and gusset plates. For sake of clearness in details, some of the connections have been drawn separately at a larger scale.

It must be remembered that two different scales are used on each half-detail diagram of each truss, so that the lengths of centre lines of members must be measured by the smaller scale, and the dimensions at the connections by the larger scale. Care must be taken to properly apply the scales.

641. Connection of X 1 and Y 1 (Figs. 601, 602, 603).—Since the maximum stress in X 1 is divided into 3 equal parts, one for the two top legs and one for each web leg, the number of single shear rivets in the member being 13 (see dimension sheet), then $\frac{13}{3} = 4\frac{1}{3}$, or say 5, say 3 in each top leg and 5 in each web leg, to transmit the stress to the gusset. Additional rivets are here added for appearance and to hold the connection together properly.

The top legs of upper chord are riveted to pieces of $5 \times 3 \times \frac{1}{2}''$ angles, which are then riveted through the web legs of chord and the gusset.

The bottom legs of lower chord are riveted to pieces of $3\frac{1}{2} \times 3 \times \frac{1}{2}''$ angles, that are then riveted through the gusset.

To bottom of gusset are riveted through 2 pieces $6 \times 6 \times \frac{1}{2}''$ angles to form a base to rest on the masonry of the wall.

642. Connection of X 1, X 3, etc. (Figs. 601, 606).—Maximum stress in 2 3 = − 3.4 tons. Two rivets are required in 1 2 and 2 3; 6 rivets in upper chord are ample. Top legs of 2 3 are connected to gusset by pieces of $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}''$ angles.

643. Connection of Y 1, Y 2, and 1 2 (Fig. 601).—Made by a gusset bar $\frac{5}{8} \times 2''$, 2 through rivets in 1 2 and 2 through the lower chord.

644. Connection of X 3, X 5, etc. (Figs. 601, 604).—Splice in upper chord and thickness of web legs reduced from $\frac{7}{16}''$ to $\frac{3}{8}''$. Maximum stress at splice = 27.9 tons, requiring 11 S. S. rivets, say 4 in top legs and 8 in web legs. Since half the stress in the web legs is transmitted directly through the gusset, and these rivets are in quadruple shear, 2 rivets in each would suffice in each web leg. But it is best to use 4, to allow for stresses produced by connecting web members there, moments at the connection, etc.

Member 3 4 is connected by pieces of $2 \times 2 \times \frac{3}{16}''$ angles; member 4 5 by pieces of $3 \times 3 \times \frac{5}{16}''$ angles. These require 2 rivets in each.

645. Connection of Y 2, Y 4, etc. (Fig. 601).—Splice in the lower chord. Maximum stress at splice = + 29.1 tons, requiring 11 S. S. rivets. Hence, as before, use $\frac{5}{16}''$ cover with 4 S. S. rivets in each end. Connections of web members as before, 2 rivets in each.

646. Connections at other Apexes.—Connection of X 5, X 7, etc. (Fig. 601).

Two rivets in each connection of web members; 6 rivets through upper chord.

Connection of Y 4, Y 6, etc. (Fig. 601).

Same arrangement as for the last connection.

Connection of X 7, X 9, etc. (Fig. 601).

Splice in upper chord. Maximum stress = − 21.1 tons, requiring 8 S. S. rivets, say 4 in each end of $\frac{5}{16}''$ cover, to allow for connection of web members. Three rivets in web legs of 8 9, 2 in other connections.

Connection of Y 6, Y 8, etc. (Fig. 601).

Splice in lower chord. Maximum stress = + 22.9 tons, requiring 9 rivets and $\frac{5}{16}''$ cover, say 4 rivets in each end. Other connection as before.

Connection of X 9, X 9′, etc. (Fig. 601).

Splice in upper chord. Maximum stress = − 17.6 tons, requiring 7 S. S. rivets. Four rivets through each end of bent $\frac{5}{16}''$ cover; 4 through each web leg; 2 rivets in each connection of 9 9′.

Connection of Y 8, Y 8′, etc. (Fig. 601).

No splice in lower chord. Connections of web members as before.

Example 4.—A Steel Fink Truss of 16 Panels

647. Remarks.—Resume Example 4 of Chapters IV and XI, a Fink truss of 16 panels and span of 128 ft., which requires expansion rolls to be provided at one end of the truss, the other end being fixed to the wall.

Two methods of construction of this truss are to be detailed here in order to make a comparison of their economy in Chapter XIV.

648. Channels and Rods; Pin Connections.—A (Fig. 607). Upper chord is composed of 2, 9″ 15# channels set 6″ apart in clear, with flanges latticed together and batten plates at each apex; web struts consist each of 2 angles riveted to gusset eye at each end; lower chord and web ties are each composed of 2 welded loop-eye steel rods, flattened to $\frac{4}{5}$ diameter at the pin. Connections are each made on a steel pin of proper diameter with pin nut on each end and hexagonal nuts, excepting for splices in the upper chord, which are made with flange and web covers on exteriors of channels. Dimensions of each member are given on the dimension sheet, but diameters of pins are to be determined by Table U, after the maximum single shear, bearing pressure per inch length, and the maximum bending moment in inch-tons have been found for each pin.

Fig. 609 shows a section through the roof parallel to middle vertical plane of the truss. The $\frac{7}{8}''$ sheathing is nailed to wooden strips $1\frac{3}{4}''$ screwed or bolted on the tops of the steel channel rafters, which are riveted through $\frac{5}{16}''$ batten plates to the upper flanges of the purlins. The latter are riveted through $\frac{5}{16}''$ batten plates to the top flanges of upper chord of the truss. Fig. 609 further represents the connection of X 6, 6 7, 7 8, 8 9, and X 9 together.

649. Connection at End of Truss (Figs. 608, 613).—Fig. 608 shows an elevation of the connection of X 1 and Y 1. The diameter of the pin must be first determined. Spacing the channels 6″ apart leaves 2″ for each rod end and the same for bearing of the pin on the cast-iron wall plate under the end of the truss. In Fig. 610 two lines are drawn parallel to centre lines of X 1 and Y 1, and on the upper line is laid off the maximum stress of − 60.8 tons, acting in the member X 1. A vertical through its end cuts the line parallel to Y 1 and scales 21.0 tons = the maximum vertical wall reaction at the connection. On the left, in Fig. 611, is shown the moment diagram for the pin caused by the two rods forming Y 1. On the right in the same figure is the moment diagram for the pin produced by the upward reaction of the support. The moment of the rods = intercept × pole distance = 0.95 × 30 = 28.5 inch-tons, and that of the middle support = 1.05 × 30 = 31.5 inch-tons. Laying off these moments in Fig. 612 parallel to the respective members, the resultant or maximum moment acting on the pin is found to be 43.00 inch-tons.

650. Diameter of Pin.—Then by Table U,

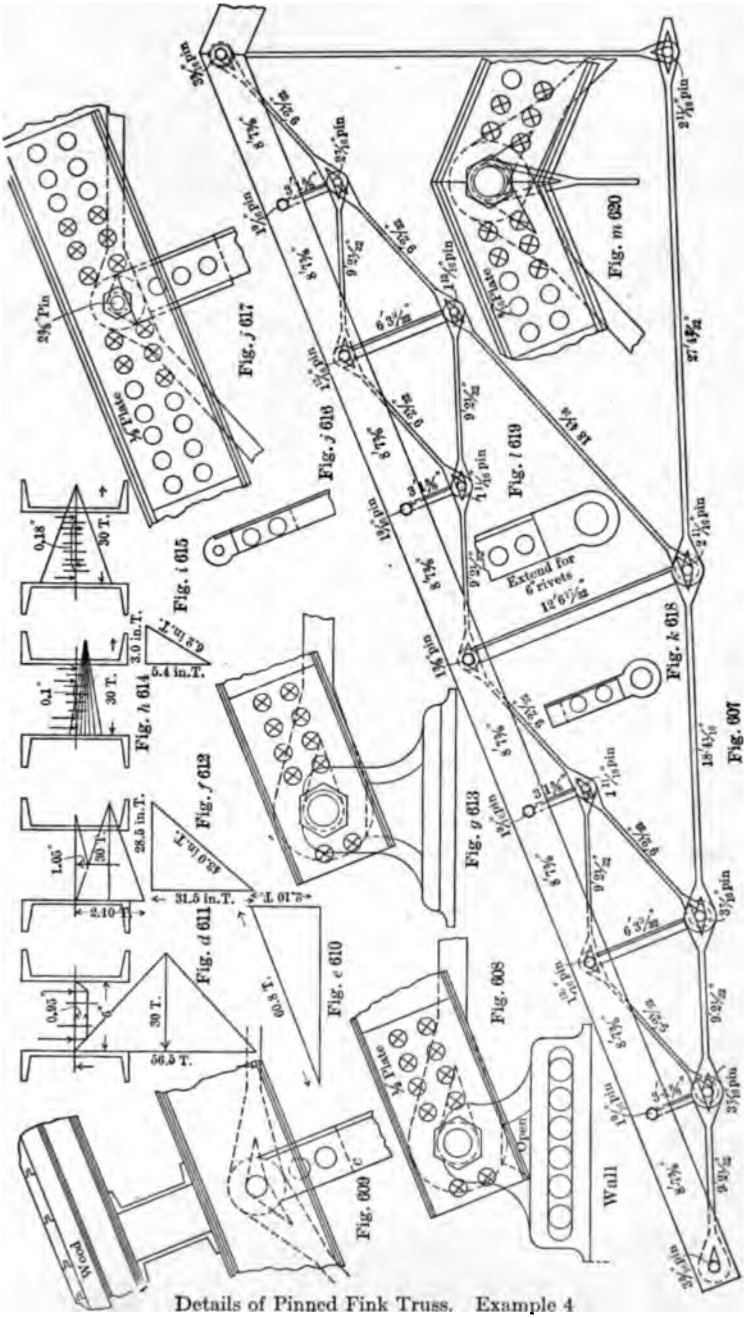
Single shear on pin = 28.25 tons; $2\frac{9}{16}''$ pin is required.

Bearing per inch length of pin = 14.13 tons; $1\frac{3}{16}''$ pin is required.

Bending moment on pin = 43.0 inch-tons; $3\frac{5}{16}''$ pin is necessary.

Therefore the pin must have a finished diameter not less than $3\frac{5}{16}''$.

The maximum stress in X 1 being − 60.8 tons, it is then 30.4 tons in each channel, which must be transmitted to the pin by the web only. The web of the channel being 0.29″ thick, the maximum safe compression transmitted to pin by the web = 0.29 × 9.0″ × 7.0 tons = 18.27 tons. Then 30.40 − 18.27 = 12.13 tons to be transmitted through a reënforcing plate on outside of web. Also 18.27 : 12.13 : : 0.29 : 0.19″ = thickness of the plate. Best make it $\frac{5}{16}''$ in order to develop full single shear of the rivets.



Details of Pinned Fink Truss. Example 4

But the maximum bearing stress on $3\frac{5}{16}''$ pin = 41.41 tons per inch length. Then 41.41 : 30.4 : : 1″ : 0.734″ = total thickness of web and plate. And 0.734″ − 0.29 = 0.444, say 1/2″ thickness of plate. This greater thickness is accordingly to be used. Also 0.29 + 0.50 = 0.79″. 0.79 : 0.50 : : 30.4 tons : 19.2 tons to be transmitted to the plate. By Table T, this requires 9, $\frac{3}{4}''$ rivets, say 10.

651. Expansion Rolls at End.—Assume 2″ diameter of expansion rolls 12″ long, between truss and wall plate.

Maximum pressure on rolls = 21.0 tons = vertical reaction.

By formula 172, $0.15\, n\, l\, d = 21.0$ tons.

Then $n = \dfrac{21.0}{0.15 \times 12 \times 2} = \dfrac{21.0}{3.6} = 5.84$, say 6 rolls, 2″ diameter and 12″ long.

The roll seat and cap are here designed in cast iron for convenience.

Fig. 613 is a similar design for the fixed support at the other end of the truss.

652. Connection of X 6, 6 7, 7 8, 8 9, and X 9.—Fig. 614 is a diagram for obtaining the maximum bending moment on pin. The maximum stress of + 11.6 tons in the two rods composing member 6 7, and likewise for member 8 9, produces a component of 4.0 tons acting parallel to 7 8 and opposed to its maximum stress of − 10.4 tons. Also a component of 11.0 tons acting perpendicular to 7 8 and opposed by an equal component of the stress in 8 9. The left part of Fig. 614 gives an intercept of 0.1″ in the direction of 7 8. The right portion has an intercept of 0.18″ perpendicular to it. Hence the corresponding moments are 3.0 and 5.4 inch-tons. Laying these moments off in Fig. 615, their resultant moment is found = 6.2 inch-tons.

Maximum single shear on pin = 5.8 tons. Pin $1\frac{3}{16}''$ diameter.

Maximum bearing = 10.4 tons on $\frac{5}{8}''$ length of pin for member 7 8 = 16.65 tons per inch length. Pin $1\frac{7}{16}''$.

Maximum bending moment = 6.2 inch-tons. Pin $1\frac{7}{8}''$.

Therefore this pin is to be $1\frac{7}{8}''$ diameter.

It is assumed that the upper chord is spliced at this connection only. Maximum stress in one channel there = -28.8 tons, which, by Table T, requires 13, $\frac{3}{4}''$ rivets in each end of cover-plate. Also $\frac{28.8}{8} = 3.6$ sq. ins. in cross-section of cover-plate, which is 7.8″ wide between the flanges. Hence its thickness $= \frac{3.6}{7.8} = 0.46''$, say $\frac{1}{2}''$, same as for plates at ends.

Stress in each angle composing member 7 8 = $-$ 5.2 tons, which requires 2 rivets through $\frac{5}{8}''$ gusset eye-plate.

The remaining connections of the members are to be worked out in the same manner, with the results indicated in Fig. 607.

653. Connections at other Apexes.—Connection of X 1, 1 2, and X 2, etc.:

Maximum single shear = 1.3 tons. $\frac{15}{16}''$ pin.

Maximum bearing = 4.5 tons per in. 1″ pin.

Maximum moment = 3.9 tons. $1\frac{9}{16}''$ pin. No reënforcement.

Connection of X 2, 3 4, X 5, etc.:

Maximum single shear = 2.6 tons. $\frac{15}{16}''$ pin.

Maximum bearing = 16.0 tons per in. $1\frac{3}{8}''$ pin.

Maximum moment = 7.1 inch-tons. $1\frac{3}{8}''$ pin. No plate.

Connection of X 14, 15 15′, and X 14′ at ridge.

Maximum single shear = 18.95 tons. $2\frac{1}{16}''$ pin.

Maximum bearing = 9.75 tons. $\frac{15}{16}''$ pin.

Maximum moment = 39.0 inch-tons. $3\frac{3}{16}''$ pin. $\frac{5}{16}''$ plate on web. $\frac{5}{16}''$ plate on outside of web. 9, $\frac{3}{4}''$ rivets in each end of plate.

Connection of Y 1, 1 2, and Y 3.

Maximum single shear = 26.8 tons. $2\frac{7}{16}''$ pin.

Maximum bearing = 15.1 tons per 1″. $1\frac{5}{16}''$ pin.

Maximum moment = 48.8 inch-tons. $3\frac{7}{16}''$ pin.

Connection of Y 3, 3 4, and Y 7.

Maximum single shear = 19.6 tons. $2\frac{1}{16}''$ pin.

Maximum bearing = 15.1 tons per 1″. $1\frac{5}{16}''$ pin.

Maximum moment = 49.5 inch-tons. $3\frac{7}{16}''$ pin.

Connection of 4 7, 5 6, and 6 7.

Maximum single shear = 3.9 tons. $\frac{15}{16}''$ pin.

Maximum bearing = 5.7 per in. $\frac{15}{16}''$ pin.

Maximum moment = 4.9 inch-tons. $1\frac{11}{16}''$ pin.

Connection of 8 9, 9 10, and 10 11.

$1\frac{11}{16}''$ pin like the last.

Connection of 8 15, 11 12, and 12 15.

Maximum single shear = 3.9 tons. $\frac{15}{16}''$ pin.

Maximum bearing = 8.8 tons per in. $\frac{15}{16}''$ pin.

Maximum moment = 8.4 inch-tons. $1\frac{15}{16}''$ pin.

Connection of 12 15, 13 14, and 14 15.

Maximum single shear = 11.9 tons. $1\frac{11}{16}''$ pin.

Since there is one row of rivets in top flange and 2 rows in web leg of each angle, one-third the stress in the member is transmitted through top legs, requiring 4 S. S. rivets in each top leg of member $X\,1$ and in each leg of piece of $5\frac{1}{2} \times 3 \times \frac{3}{8}''$ angle inserted between legs of angle to transmit stress in top legs to gusset. The same number is required through web legs and gusset, the rivets there being in double shear. A larger number of rivets are here used in order to make a good connection to gusset.

For similar reasons, 4 rivets are used in each bottom leg of $Y\,1$, and in each leg of piece of $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}''$ angle connecting bottom leg and gusset. Same number of rivets through web legs and gusset, being in double shear. This number is here increased to firmly connect the member with gusset.

656. Connections at Other Apexes.—Connection of $X\,1$ and $X\,2$; $X\,5$ and $X\,6$; $X\,9$ and $X\,10$; $X\,13$ and $X\,14$ (Figs. 621, 624).

Since the members 1 2, etc., are but $3'\,1\frac{5}{8}''$ in centre length and are each subjected to a stress of $-$ 2.6 tons, a gusset bar $\frac{5}{8}'' \times 2''$ with 2 rivets in each end will be ample without connecting both legs of the angles.

Connection of $Y\,1$, $Y\,3$, etc. (Figs. 621, 625). Two rivets in each connection of web members and 4 in lower chord are ample.

Connection of $X\,2$, $X\,5$, etc.; $X\,10$, $X\,13$, etc. (Fig. 621). No splice in upper chord here. Two rivets in each web connection and 6 in upper chord are ample.

Connection of $Y\,3$, $Y\,7$, etc. (Fig. 621). Splice in lower chord. Maximum stress = + 52.7 tons; 20 S. S. rivets are required.

Hence use 8 S. S. rivets in each end of $\frac{5}{16}''$ cover and top legs of angles; 4 rivets in quadruple shear through each end of web covers, web legs of angles and gusset; 2 rivets in each web connection.

Connection of 4 7 and 6 7, etc.; 8 9 and 8 11, etc. (Fig. 621). Two rivets are required in each connection of these members.

Connection of $X\,7$ and $X\,8$, etc. (Figs. 621, 623). Splice in upper chord. Maximum stress = $-$ 57.6 tons; 22 S. S. rivets are required. Then 8 S. S. rivets are to be used in each end of top

cover, and 4 Q. S. rivets in each end of web covers, through covers, web legs, and gusset; 2 rivets in each web connection.

Connection of Y 7 and Y 15, etc. (Fig. 621). Splice in lower chord. Maximum stress = + 44.9 tons; 17 S. S. rivets are required.

Hence use 6 rivets in each end of bottom cover and 4 Q. S. rivets through web covers, legs, and gusset.

Connection of 8 15, 12 15, etc. (Fig. 621). Splice. Maximum stress = + 23.1 tons; requires 9 S. S. rivets.

Then 3 S. S. rivets in each end of $\frac{5}{8}''$ bottom cover; 2 Q. S. rivets through web covers, legs, and gusset.

Connection of Y 14 and Y 14′, etc. (Figs. 621, 626). Maximum stress = + 53.6 tons; 21 S. S. rivets are required.

Then 8 S. S. rivets in each end of $\frac{5}{16}''$ bottom cover; 8 D. S. rivets through each end of web cover, legs, and gusset; 3 S. S. rivets in each web connection, except in 15 15′, where 2 rivets suffice for a stress of + 1.9 tons in the member. No web covers are here used on account of the awkward shape, it being more economical to increase the area of gusset and use more rivets.

EXAMPLE 5.—A STEEL SEMICIRCULAR CRESCENT TRUSS

Resume the stress and dimension sheet obtained for this example in Chapter XI.

657. A.—All members of the truss are straight (Fig. 627). Therefore the chords must be spliced at each apex. The purlins are of wood and their ends may be blocked out to bring the sheathing into the required curve as in Fig. 628, or their ends may be boxed down on the chord for the same purpose as in Fig. 629, which is simplest and best. They may be fastened on the chord by lagscrews and brackets attached to the top legs. The roof will appear best if all purlins are made 4 × 10″, set to form 4 equal spaces per panel.

Fig. 627 is an elevation of one-half the truss, both ends being bolted down to the walls. It will be most convenient here to omit web covers at the apex splices.

658. Connection at End of Truss.—Connection of X 1 and Y 1 (Figs. 627, 630). Maximum stress in X 1 = − 41.4 tons; 16 S. S. rivets; in Y 1 = + 30.5 tons; 12 S. S. rivets.

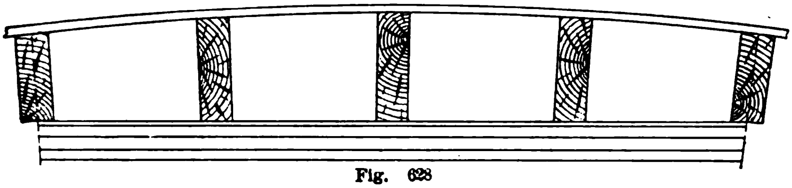


Fig. 628

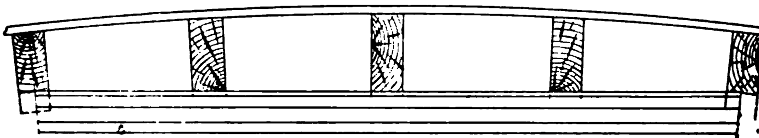
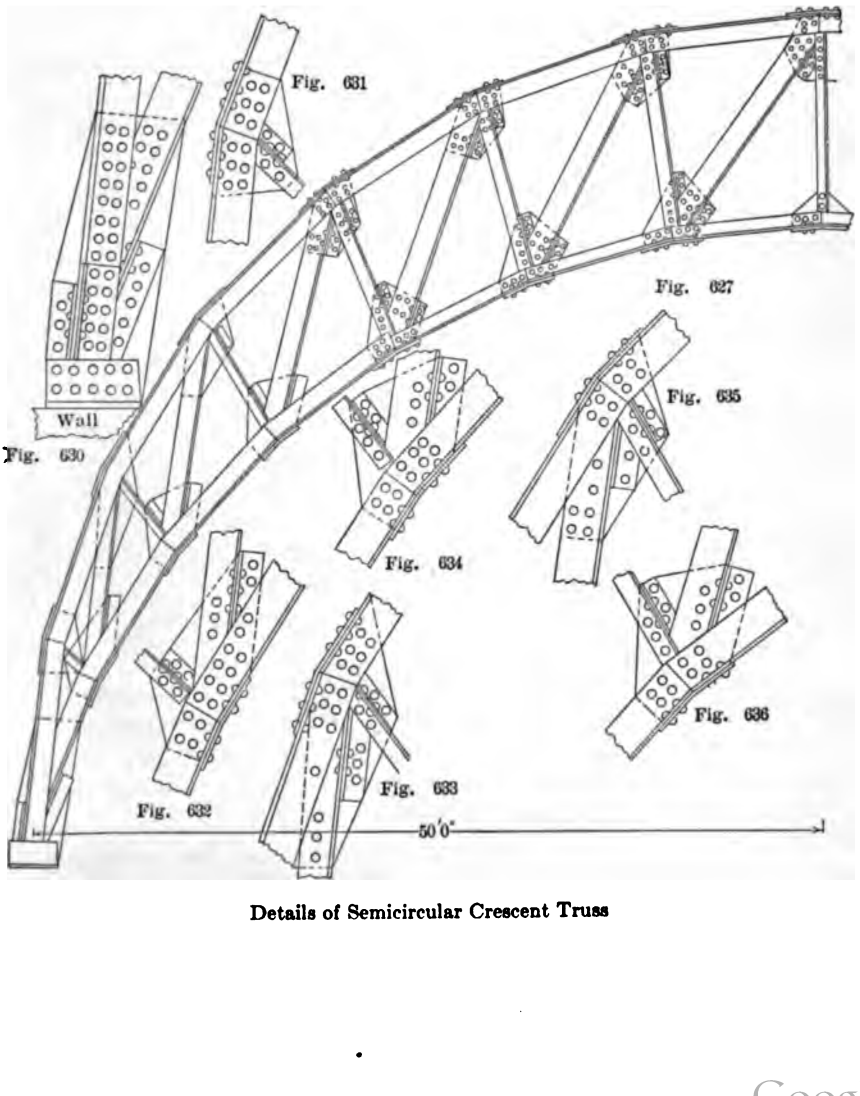


Fig. 629



Details of Semicircular Crescent Truss

This requires 3 S. S. rivets in each leg of piece of $3\frac{1}{2} \times 3 \times \frac{3}{8}''$ angle connecting top leg of upper chord with gusset. Also 2 S. S. rivets through each leg of piece of $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}''$ angle connecting bottom legs of lower chord with gusset. Also same number of D. S. rivets through each web leg of upper and lower chords and gusset. A larger number are inserted here in order to make a firm connection with the gusset. The foot is formed by riveting one $6 \times 6 \times \frac{3}{4}''$ angle on each side of the gusset.

659. Connections at other Apexes.—Connection of X 1, X 2, etc. (Figs. 627, 631). Maximum stress = − 41.4 tons; 16 S. S. rivets.

Then 6 S. S. rivets through end of top cover; 6 D. S. rivets through web legs and gusset; 2 S. S. rivets in each web connection.

Connection of Y 1, Y 3, etc. (Figs. 627, 632). Maximum stress = + 30.5 tons. Then 4 S. S. rivets in each end of bottom cover; 4 D. S. rivets through web legs and gusset; 2 S. S. rivets in each web connection.

Connection of X 2, X 4, etc. (Figs. 627, 633). Maximum stress = − 37.1 tons; 14 S. S. rivets. Then same number of rivets in each connection as for X 1, X 2, etc.

Connection of Y 3, Y 5, etc. (Figs. 627, 632). Maximum stress = + 28.4 tons; 11 S. S. rivets. Then 4 S. S. rivets in each end of top cover; 4 D. S. rivets through web legs and gusset; 2 S. S. rivets in each web connection.

Connection of X 4, X 6, etc. (Figs. 627, 635). Maximum stress = − 33.8 tons; 13 S. S. rivets. Hence 4 S. S. rivets in each end of bottom cover; 4 D. S. rivets through web legs and gusset; 2 S. S. rivets in each web connection.

Connection of Y 5, Y 7, etc. (Figs. 627, 636). Maximum stress = + 25.7 tons; 10 S. S. rivets. Rivets same as for the last connection.

Connection of X 6, X 8, etc. Maximum stress = − 30.7 tons; 12 S. S. rivets. Rivets same as for connection of X 4, X 6, etc.

Connection of Y 7, Y 9, etc. (Fig. 627). Maximum stress = + 21.8 tons; 9 S. S. rivets. Use 4 S. S. rivets in each end of cover; 4 D. S. rivets in Y 7 and 3 in Y 9; 2 S. S. rivets in each web connection.

Connection of X 8, X 10, etc. (Fig. 627). Maximum stress = − 26.6 tons; 11 S. S. rivets. Same number of rivets as in last connection.

Connections at remaining apexes. Use 4 S. S. rivets in each end of cover; 3 D. S. rivets through web legs and gusset; 2 S. S. rivets for each web connection.

660. B.—These connections are made in the manner described in Arts. 657–659, excepting H. Splices are not required at each apex.

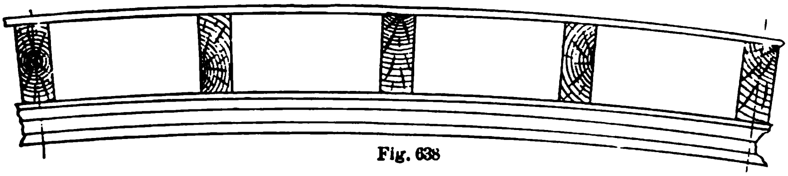


Fig. 638

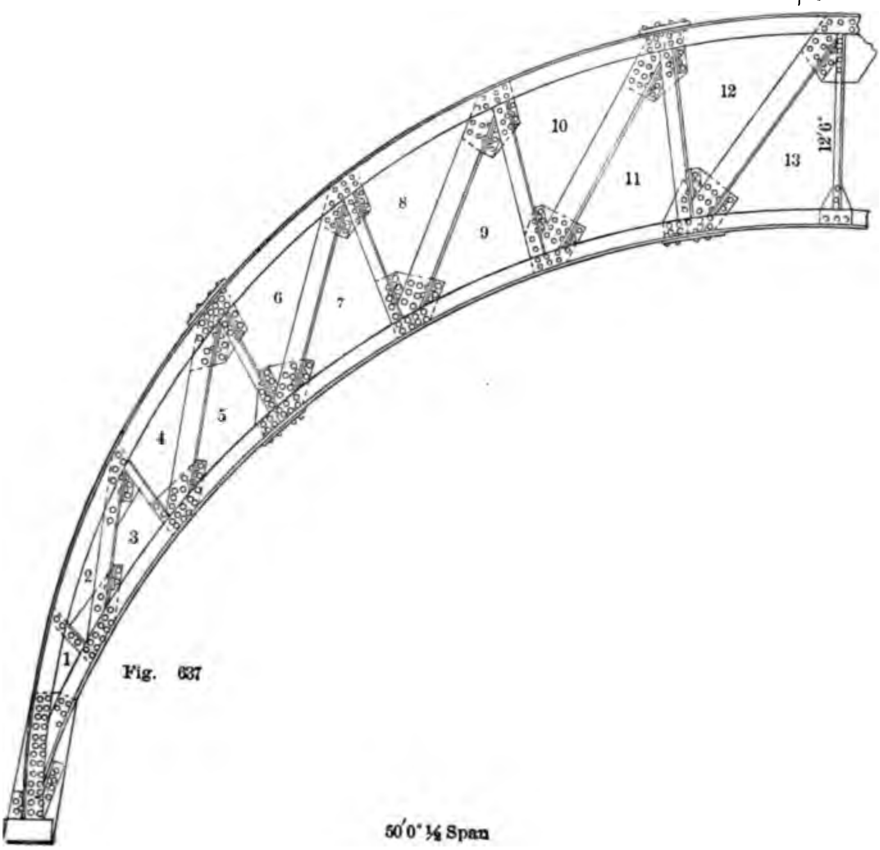


Fig. 637

50′0″ ½ Span

Details of Semicircular Crescent Truss

CHAPTER XIII

DEFORMATION OF TRUSSES

661. General Remarks.—Whatever care and precision are employed in designing and constructing a roof truss, it will be found to change its form somewhat after its erection. This alteration in form results from several causes, which frequently combine to affect the shape of the truss, and they should be carefully considered, though too frequently neglected. They are the following:

1. Effect of accidental and extra loads.
2. Inaccuracies in construction and looseness of connections.
3. Shrinkage of timbers, usually small lengthwise.
4. Shortening of members under compression.
5. Lengthening of members under tension.
6. Shortening of members exposed to extreme cold.
7. Lengthening of members exposed to extreme heat.

The first two causes may be nearly eliminated by great care in construction and erection.

The third may frequently be remedied by screwing up nuts on bolts and rods, thus reducing their lengths.

The remaining causes of change of form cannot be avoided, and they should always be carefully considered in case of trusses of wide spans, especially if exposed to high or low temperatures. Stress and temperature changes may be similar or opposed in each member, *i.e.*, may be cumulative or may partly neutralize each other.

When trusses are without hinges and are firmly riveted at all connections, temperature variations may also considerably increase the stresses in some of the members, which produces additional changes in their lengths.

662. Deflection of Truss.—Deflection of a truss at its mid-span is usually visible and is objectionable in appearance, because apparently evidence of weakness in construction. Therefore to prevent deflection below a horizontal line, the truss is often cambered, so that the deflection after erection may not bring it below the horizontal line. This is best effected by slightly increasing the lengths of members in compression and reducing those of members in tension, so as to accurately compensate for the changes in length produced

by the stresses and also for looseness at the connections. Empirical formulas for this purpose are often given and may suffice in practice for trusses of moderate span, but it is much preferable to be able to accurately determine these changes in lengths of members, so that it then becomes possible to apply the necessary corrections corresponding thereto, thus bringing the truss to the desired form after erection and the assuming of its loads.

663. Effect of Changes in Temperature.—Large structures erected in very hot or very cold climates may be laid out at the site with standard steel tapes, corrected for temperature to their standard length at 60° F. Hence if the trusses for such a building were framed and constructed in a moderate climate, say at 70° F., the lengths of the truss members require correction in order to be of accurate lengths at the site of the building. Otherwise a wide-span roof truss would not fit accurately, being too long in a hot climate or too short in a cold one.

Of course if the structure were laid out at the site with a steel tape not corrected for local temperature, these temperature corrections in lengths of truss members would not be required.

664. Extension Caused by Stress.—This is negative or — in a member under compression, positive or + in one under tension.

Let l = length of member in inches.
A = area of its cross-section in sq. ins.
Δ = − or + extension in ins.
E' = modulus of elasticity of material in lbs.
S' = − or + stress in member in lbs.

Then $\Delta = \mp \frac{S'.l}{E'A}$ = − or + extension of member in ins.

This formula will be more convenient for our purpose, if lengths of members are changed from inches to feet, and if S' and E' are changed from lbs. to tons. Substitute 2,000 S for S', 2,000 E for S', and 12 L for l, and simplify the equation.

$$\Delta = \frac{2000\, S \times 12\, L}{2000\, E \times A} = \mp \frac{12\, S\, L}{E\, A} = - \text{ or } + \text{ extension of member in ins.}$$

665. Extension Caused by Temperature Change.—The average maximum and minimum temperatures at any locality may be obtained from a good isothermal chart. Since these are usually given in Centigrade temperatures, these must be changed for our use into Fahrenheit temperatures.

For example, according to Andree's "Handatlas" (edition of 1906), the average temperature at Chicago is + 24.8° Fah. in January, 76.1° in July, 51.3° F. for the year. Average temperature at Panama is 66.2° in January, 95° in July, or 89.8° for the year.

Hence if a structure were framed in Chicago at a temperature of 60° F., and were to be erected at Panama in July, it would be necessary to make a correction for a difference of temperature of at least 35° F., if the building were laid out at the site with a steel tape corrected to the standard temperature of 60° F.

Let F° = number of degrees difference in temperature, usually taken from 60°.

L = length of member in feet.

For wrought iron, extension = + or − .00007776 L, per deg. F. in ins.

For steel, extension = + or − .00008288 L, in ins.

For wood, extension = + or − .00003312 L, in ins.

These temperature extensions are usually neglected for small differences in temperature, but they should be considered in case of large structures.

666. Calculation of Stress Extensions.—

EXAMPLE 2.—A WOODEN TRUSS

Resume Example 2 of Chapter XI, a wooden truss with steel verticals.

a. Members in compression.

Member X 1: stress − 42.2 tons; C. L. 10.7708′; wood; $= .0909'' = \frac{3}{32}''$.

$$\Delta = \frac{12\,S\,L}{600\,A} = \frac{S\,L}{50\,A} = \frac{42.2 \times 10.7708}{50 \times 100} = 0.0909'' = \frac{3}{32}''.$$

Mem. X 3; stress − 36.4 tons; C. L. as before; wood; $-.0784 = \frac{5}{64}''$

Mem. X 5; stress − 31.7 tons; C. L. as before; wood; $-.0683 = \frac{1}{16}''$

Mem. X 7; stress − 26.9 tons; C. L. as before; wood; $-.0579 = \frac{1}{16}''$

Mem. X 9; stress − 22.1 tons; C. L. as before; wood; $-.0476 = \frac{3}{64}''$

Mem. 2 3; stress − 5.1 tons; C. L. as before; wood; $-.0305 = \frac{1}{32}''$

Mem. 4 5; stress − 6.2 tons; C. L. 12.8073′; wood; $-.0441 = \frac{3}{64}''$

Mem. 6 7; stress − 7.6 tons; C. L. 15.6198′; wood; $-.0495 = \frac{3}{64}''$

Mem. 8 9; stress − 9.2 tons; C. L. 18.867 2′; wood; $-.0542 = \frac{1}{16}''$

b. Members in tension.

Mem. Y 1; stress + 39.5 tons; C. L. 10′; wood; $+.0494 = \frac{3}{64}''$

Mem. Y 2; stress + 39.5 tons; C. L. 10′; wood; $+.0494 = \frac{3}{64}''$

Mem. Y 4; stress + 34.7 tons; C. L. 10′; wood; $+.0434 = \frac{3}{64}''$

Mem. Y 6; stress + 30.0 tons; C. L. 10′; wood; $+.0375 = \frac{1}{32}''$

Mem. Y 8; stress + 25.0 tons; C. L. 10′; wood; $+.0313 = \frac{1}{32}''$

Mem. 1 2; stress + 0.0 tons; C. L. 4′; steel; $+.0000 = 0''$

Mem. 3 4; stress + 2.2 tons; C. L. 8′; steel; $+.0674 = \frac{1}{16}''$

Mem. 5 6; stress + 3.9 tons; C. L. 12′; steel; $+.0747 = \frac{5}{64}''$

Mem. 7 8; stress + 5.9 tons; C. L. 16′; steel; $+.0995 = \frac{3}{32}''$

Mem. 9 9′; stress + 13.2 tons; C. L. 20′; steel; $+.1236 = \frac{1}{8}''$

It is unnecessary to change the lengths of the vertical rods, since these may be adjusted by the nuts.

667. Application of these Results.—It at once becomes evident, that if these corrections be made in framing the lengths of the members, by increasing those in compression and reducing those in tension, by the computed extensions for each, the truss will have the intended form without deflection, when it is erected and has received the loads assumed in designing it.

The length of each principal is thus increased by $\frac{3}{32} + \frac{5}{64} + \frac{1}{16} + \frac{1}{16} + \frac{3}{64} = \frac{22}{64}'' = \frac{11}{32}''$.

The length of the entire lower chord is to be reduced by $2\left(\frac{3}{64}+\frac{3}{64}+\frac{3}{64}+\frac{1}{32}+\frac{1}{32}\right)=\frac{12}{32}''$.

668. Calculation of Temperature Extensions.—Assume that in Champaign the temperature may range from 0° to 100° Fah., within a structure for warehouse purposes, not regularly heated in winter. From the standard temperature of 60° this is a − variation of 60° and a + variation of 40°.

We will apply this to the same example.

Member X 1: C. L. 10.7708′; wood; $-.0214''=\frac{1}{64}''$ and $+0.143''=\frac{1}{64}''$.

By formula 176, $10.7708\times-.00003312\times60°=.0214=\frac{1}{64}''$.

By formula 176, $10.7708\times-.00003312\times40°=.0143=\frac{1}{64}''$.

Same temperature extensions in all members of upper chord.

Member Y 1: C. L. 10′; wood; $-.0199''=\frac{1}{14}''$ and $+.0133''=\frac{1}{64}''$.

Same temperature changes in all members of lower chord.

Mem. 2 3: C. L. 10.7708′; wood; $-.0214''=\frac{1}{64}''$ and $+.0143''=\frac{1}{64}''$.

Mem. 4 5: C. L. 12.8073′; wood; $-.0254''=\frac{1}{32}''$ and $+.0169''=\frac{1}{64}''$.

Mem. 6 7: C. L. 15.6198′; wood; $-.0310''=\frac{1}{32}''$ and $+.0207''=\frac{1}{64}''$.

Mem. 8 9: C. L. 18.8672′; wood; $-.0375''=\frac{1}{32}''$ and $+.0250''=\frac{1}{32}''$.

Mem. 1 2: C. L. 4.′; steel; $-.0198''=\frac{1}{64}''$ and $+.0132''=\frac{1}{64}''$.

Mem. 3 4: C. L. 8.′; steel; $-.0397''=\frac{3}{64}''$ and $+.0265''=\frac{1}{32}''$.

Mem. 5 6: C. L. 12″; steel; $-.0595''=\frac{1}{16}''$ and $+.0330''=\frac{1}{32}''$.

Mem. 7 8: C. L. 16.′; steel; $-.0794''=\frac{5}{64}''$ and $+.0530''=\frac{3}{64}''$.

Mem. 9 9′: C. L. 20.′; steel; $-.0992''=\frac{3}{32}''$ and $+.0661''=\frac{1}{16}''$.

Adjustments for the rods can be made by the nuts.

These results are collected in the following extension sheet.

669. Extension Sheet.—

Member.	*M*-stress.	*C*-length.	Stress ext.	$-60°$ ext.	$+40°$ ext.
X 1	-42.2	10.7708	$-\frac{3}{32}''$	$-\frac{1}{64}''$	$+\frac{1}{64}''$
X 2	-36.4	10.7708	$-\frac{5}{64}''$	$-\frac{1}{64}''$	$+\frac{1}{64}''$
X 5	-31.7	10.7708	$-\frac{1}{16}''$	$-\frac{1}{64}''$	$+\frac{1}{64}''$
X 7	-26.9	10.7708	$-\frac{3}{64}''$	$-\frac{1}{64}''$	$+\frac{1}{64}''$
X 9	-22.1	10.7708	$-\frac{3}{64}''$	$-\frac{1}{64}''$	$+\frac{1}{64}''$
Y 1	$+39.5$	10.	$+\frac{3}{64}''$	$-\frac{1}{64}''$	$+\frac{1}{64}''$
Y 2	$+39.5$	10.	$+\frac{3}{64}''$	$-\frac{1}{64}''$	$+\frac{1}{64}''$
Y 4	$+34.7$	10.	$+\frac{1}{32}''$	$-\frac{1}{64}''$	$+\frac{1}{64}''$
Y 6	$+30.0$	10.	$+\frac{1}{32}''$	$-\frac{1}{64}''$	$+\frac{1}{64}''$
Y 8	$+25.0$	10.	$+\frac{1}{32}''$	$-\frac{1}{64}''$	$+\frac{1}{64}''$
1 2	$+\ 0.0$	4.	. . .	$-\frac{1}{64}''$	$+\frac{1}{64}''$
3 4	$+\ 2.2$	8.	$+\frac{1}{16}''$	$-\frac{3}{64}''$	$+\frac{1}{32}''$
5 6	$+\ 3.9$	12.	$+\frac{5}{64}''$	$-\frac{1}{16}''$	$+\frac{1}{32}''$
7 8	$+\ 5.9$	16.	$+\frac{3}{32}''$	$-\frac{5}{64}''$	$+\frac{3}{64}''$
9 9′	$+13.2$	20.	$+\frac{1}{8}''$	$-\frac{3}{32}''$	$+\frac{1}{16}''$
2 3	$-\ 5.1$	10.7708	$-\frac{1}{32}''$	$-\frac{1}{64}''$	$+\frac{1}{64}''$
4 5	$-\ 6.2$	12.8073	$-\frac{3}{64}''$	$-\frac{1}{32}''$	$+\frac{1}{64}''$
6 7	$-\ 7.6$	15.6198	$-\frac{3}{64}''$	$-\frac{1}{32}''$	$+\frac{1}{64}''$
8 9	$-\ 9.2$	18.8672	$-\frac{1}{16}''$	$-\frac{1}{32}''$	$+\frac{1}{32}''$

B. Example 3 of Chapter XI. A Steel Truss

Resume Example 3 of Chapter XI.

670. Stress Extensions.—Apply Formula 173 as before.

Mem. X 1; stress − 41.4 tons; C. L. 10.7708′; steel; section 8.00; − .0451″.
Mem. X 3; stress − 35.7 tons; C. L. 10.7708 ; steel; section 8.00; − .0389″.
Mem. X 5; stress − 31.3 tons; C. L. 10.7708 ; steel; section 6.10; − .0441″.
Mem. X 7; stress − 26.4 tons; C. L. 10.7708; steel; section 6.10; − .0374″.
Mem. X 9; stress − 21.7 tons; C. L. 10.7708; steel; section 5.12; − .0369″.
Mem. 2 3; stress − 5.0 tons; C. L. 10.7708; steel; section 2.88; − .0151″.
Mem. 4 5; stress − 6.1 tons; C. L. 12.8073; steel; section 4.18; − .0155″.
Mem. 6 7; stress − 7.5 tons; C. L. 15.6198; steel; section 8.72; − .0189″.
Mem. 8 9; stress − 9.1 tons; C. L. 18.8672; steel; section 8.72; − .0163″.
Mem. Y 1; stress + 38.8 tons; C. L. 10.; steel; section 7.06; + .0455″.
Mem. Y 2; stress + 38.8 tons; C. L. 10.; steel; section 7.06; + .0455″.
Mem. Y 4; stress + 34.1 tons; C. L. 10.; steel; section 7.06; + .0400″.
Mem. Y 6; stress + 29.5 tons; C. L. 10.; steel; section 7.06; + .0346″.
Mem. Y 8; stress + 24.5 tons; C. L. 10.; steel; section 5.12; + .0396″.
Mem. 1 2; stress + 0.0 ton; C. L. 4.; steel; section 1.44; + .0000″.
Mem. 3 4; stress + 2.0 tons; C. L. 8.; steel; section 1.44; + .0092″.
Mem. 5 6; stress + 3.8 tons; C. L. 12.; steel; section 1.44; + .0262″.
Mem. 7 8; stress + 5.9 tons; C. L. 16.; steel; section 1.44; + .0543″.
Mem. 9 9′; stress + 12.9 tons; C. L. 20.; steel; section 2.64; + .0809″.

671. Temperature Extensions.—Apply same formula as before, for steel.

Member X 1: C. L. 10.7708; $-$ 60°, $-$ .0534″; + 40°, + .0356″.

Same extensions for other members of upper chord.

Member Y 1: C. L. 10.; $-$ 60° $-$ .0496″; + 40°, + .0331″.

Same extensions for other members of lower chord.

Mem. 2 3: C. L. 10.7708; $-$ 60°, $-$ .0534″; + 40°, + .0356″.
Mem. 4 5: C. L. 12.8073; $-$ 60°, $-$ .0635″; + 40°, + .0423″.
Mem. 6 7: C. L. 15.6198; $-$ 60°, $-$ .0775″; + 40°, + .0517″.
Mem. 8 9: C. L. 18.8672; $-$ 60°, $-$ .0936″; + 40°, + .0624″.
Mem. 1 2: C. L. 4. $-$ 60°, $-$ .0198″; + 40°, + .0132″.
Mem. 3 4: C. L. 8. $-$ 60°, $-$ .0397″; + 40°, + .0265″.
Mem. 5 6: C. L. 12. $-$ 60°, $-$ .0595″; + 40°, + .0397″.
Mem. 7 8: C. L. 16. $-$ 60°, $-$ .0794″; + 40°, + .0529″.
Mem. 9 9: C. L. 20. $-$ 60°, $-$ .0992″; + 40°, + .0661″.

672. Extension Sheet.—

Member.	M-stress.	C-length.	Stress ext.	$-60°$ ext.	$+40°$ ext.
X 1	−41.4	10.7708	$-\frac{3}{64}''$	$-\frac{3}{64}''$	$+\frac{1}{32}''$
X 3	−35.7	10.7708	$-\frac{3}{64}''$	$-\frac{3}{64}''$	$+\frac{1}{32}''$
X 5	−31.1	10.7708	$-\frac{3}{64}''$	$-\frac{3}{64}''$	$+\frac{1}{32}''$
X 7	−26.4	10.7708	$-\frac{1}{32}''$	$-\frac{3}{64}''$	$+\frac{1}{32}''$
X 9	−21.7	10.7708	$-\frac{1}{32}''$	$-\frac{3}{64}''$	$+\frac{1}{32}''$
Y 1	+38.8	10.	$+\frac{3}{64}''$	$-\frac{3}{64}''$	$+\frac{1}{32}''$
Y 2	+38.8	10.	$+\frac{3}{64}''$	$-\frac{3}{64}''$	$+\frac{1}{32}''$
Y 4	+34.1	10.	$+\frac{3}{64}''$	$-\frac{3}{64}''$	$+\frac{1}{32}''$
Y 6	+29.5	10.	$+\frac{1}{32}''$	$-\frac{3}{64}''$	$+\frac{1}{32}''$
Y 8	+24.5	10.	$+\frac{3}{64}''$	$-\frac{3}{64}''$	$+\frac{1}{32}''$
1 2	+ 0.0	4.	+0	$-\frac{1}{64}''$	$+\frac{1}{64}''$
3 4	+ 2.0	8.	$+\frac{1}{64}''$	$-\frac{3}{64}''$	$+\frac{3}{64}''$
5 6	+ 3.8	12.	$+\frac{1}{32}''$	$-\frac{1}{16}''$	$+\frac{3}{64}''$
7 8	+ 5.9	16.	$+\frac{1}{16}''$	$-\frac{5}{64}''$	$+\frac{3}{64}''$
9 9′	+12.9	20.	$+\frac{5}{64}''$	$-\frac{7}{64}''$	$+\frac{1}{16}''$
2 3	− 5.0	10.7708	$-\frac{1}{64}''$	$-\frac{3}{64}''$	$+\frac{1}{32}''$
4 5	− 6.1	12.8073	$-\frac{1}{64}''$	$-\frac{1}{16}''$	$+\frac{3}{64}''$
6 7	− 7.5	15.6198	$-\frac{1}{64}''$	$-\frac{5}{64}''$	$+\frac{3}{64}''$
8 9	− 9.1	18.8672	$-\frac{1}{64}''$	$-\frac{3}{32}''$	$+\frac{1}{16}''$

The method of computing the magnitudes of temperature changes in the lengths of truss members has been sufficiently explained to make it easily applicable to more complicated types of trusses.

CHAPTER XIV

WEIGHTS OF ROOF TRUSSES

673. General Remarks.—It is frequently necessary to make an accurate estimate of the weight of a roof truss, especially if entirely constructed of steel, in order to determine its cost. The weights of the members and connections are then computed and entered on a preliminary weight sheet. Should the total weight of the truss materially differ from its assumed weight by formula, employed in computing the P loads at the apexes, there should be a revision of the P loads, P stresses, maximum stresses, sectional dimensions of members in accordance with the preliminary total weight. The weights of the members are then revised, and the results of revision are entered on the revised weight sheet. A single revision is usually sufficient, though it may be repeated if necessary. The revised weights may then be safely taken as the actual weight of the truss. This method is here applied to several examples of trusses, already dimensioned in Chapter XI, and detailed in Chapter XII.

Example 2.—A Wooden Truss

674. Description.—Resume Example 2 of Chapter XI, which is the same as Example 1 of Chapter IV, excepting that the crane and ceiling loads are omitted, and connections are as detailed in Fig. 596, Art. 633, of Chapter XII. Note that this half elevation of the truss is drawn at two different scales, one for the truss diagram only composed of the centre lines of the members, the other being much larger and to be used for the details of connections alone at an apex.

675. Calculation of Weights.—Timbers are here of shortleaf pine at 36 # per cu. ft.; steel plates and rods at 489.6 # per cu. ft., as given in Cambria and Carnegie; cast-iron washers are taken at 450 # per cu. ft. Weights of steel plates and bolts are taken from Cambria and Carnegie; those of rods, upset ends, nuts, and cast-iron washers are given in Tables V and W. It is most convenient to take centre lengths of members in ft. and hundredths, and the C. L. weights of members should be entered separately from the weights of connections at apexes; also the weights of the different materials

should be separated. The weight sheet will then contain data very useful in making rapid computations for similar trusses in future. The dimensions of members are here taken from the dimension sheet for this truss, Art. 570, Chapter XI.

A. C.L. Weights of Members.

676. Wooden Members.—

Mem. X 1: C. L. 10.77′; 10 × 10″; 25 # per ft.; wt. = 10.77 × 25 = 269.3 #.
Members X 3, X 5, X 7, and X 9 each have the same weights.
Mem. Y 1: C. L. 10.00′; 10 × 12″; 30 # per ft.; wt. = 10.00 × 30 = 300.0 #.
Members Y 2, Y 4, Y 6, and Y 8 each have the same weights.
Mem. 2 3: C. L. 10.77′; 6 × 6″; 9 # per ft.; wt. = 10.77 × 9 = 96.9 #
Mem. 4 5: C. L. 12.81′; 6 × 6″; 9 # per ft.; wt. = 12.81 × 9 = 115.3 #.
Mem. 6 7: C. L. 15.62′; 6 × 8″; 12 # per ft.; wt. = 15.62 × 12 = 187.4 #.
Mem. 8 9: C. L. 18.87′; 8 × 8″; 16 # per ft.; wt. = 18.87 × 16 = 301.9 #.

677. Steel Members.—

Mem. 1 2: C. L. 4.00′; 1, $\frac{1}{2}$″ rod; 0.67 # per ft.; wt. = 4.00 × 0.67 = 2.7 #.

Mem. 3 4: C. L. 8.00′; 1, $\frac{1}{2}$″ rod; 0.67 # per ft.; wt. = 8.00 × 0.67 = 5.4 #.

Mem. 5 6: C. L. 12.00′; 1, $\frac{11}{16}$″ $R.$; 1.26 # per ft.; wt. = 12.00 × 1.26 = 15.1 #.

Mem. 7 8: C. L. 16.00′; 1, $\frac{13}{16}$″ $R.$; 1.76 # per ft.; wt. = 16.00 × 1.76 = 28.2 #.

Mem. 9 9′: C. L. 20.00′; 1, $\frac{15}{16}$″ $R.$; 4.60 # per ft.; wt. = 20.00 × 4.60 = 92.0 #.

B. Weights of Connections.

678. Wooden Members.—The lower end of X 1 is assumed to be cut off to fit on top of Y 1, which extends one foot beyond each end apex to afford bearing of wall. The loss or increase in length of a wooden member is measured on its centre line.

Mem. X 1: deduct 1.8′; 25 # per ft.; deduction = 1.8 × 25 # = 45.0 #.
Members X 3, Y 5, X 7, X 9 require no deductions at apexes.
Mem. Y 1: add 1.00′; 30 # per ft.; addition = 1.00 × 30.0 = 30.0 #.
Members Y 2, Y 4, Y 6, Y 8 require no deductions at apexes.
Mem. 2 3: deduct 2.44′; 9 # per ft.; deduction = 2.44 × 9 = 22.0 #.
Mem. 4 5: deduct 1.57′; 9 # per ft.; deduction = 1.57 × 9 = 14.1 #.
Mem. 6 7: deduct 1.36′; 12 # per ft.; deduction = 1.36 × 12 = 16.4 #.
Mem. 8 9: deduct 1.27′; 16 # per ft.; deduction = 1.27 × 16 = 20.3 #.

679. Steel Rods, Upset Ends, Nuts, Cast-iron Washers (Art. 635).—The additional length of each steel rod = sum of distances at

ends from apex to nearest end of upset. To its weight is to be added the weights of two upset ends, two nuts, and two cast-iron washers, the latter being entered in a separate column of the preliminary weight sheet. The ends of rod 1 2 are not upset, since the stress on it = 0.0 ton in stress sheet.

Member 1 2: add 1.50′ ft. × 0.67 # × 1.0 #; 2 nuts = 0.17 # by Table W; 2 C. I. washers = 1.41 # by Table W. Addition for steel = 1.0 + 0.17 = 1.17 #; for C. I. 1.41 #.

Member 3 4: add 0.76′ × 0.67 # = 0.51 #; by Table V, 2 upset ends = 1.06 #; 2 nuts = 0.63 #; 2 C. I. washers = 3.45 #. Add for steel 0.51 + 1.68 + 0.63 = 2.20 #; for C. I. 3.45 #.

Member 5 6: add 0.76′ × 1.26 # = 0.96 #; 2 upsets = 1.99 # by Table V; 2 nuts = 1.42 #. Add for steel 0.96 + 1.99 + 1.42 = 4.37 #; 2 C. I. washers = 8.42 # by Table V.

Member 7 8: add 0.90′ × 1.76 # = 1.58 #; 2 upset ends = 2.76 #; 2 nuts = 2.04 #. Add for steel 1.58 + 2.67 + 2.04 = 6.30 #; 2 C. I. washers = 12.82 #.

Member 9 9′: add 0.93′ + 4.60 = 4.28 #; 2 upsets = 7.15 #; 2 nuts = 11.11 #. Add for steel 4.28 + 7.15 + 11.11 = 22.50 #; 2 C. I. washers = 55.2 #.

680. Splices in Chords.—Connection of X 1 and Y 1.

2, ⅜″ fish-plates, 6.01 sq. ft. each × 15.3 # =	183.9 #
16, 1″ × 15″ bolts × 4.27 # each =	68.3 #
2, ¾″ × 15″ bolts × 2.21 # each =	4.4 #
Total steel =	256.6 #

Splice in Y 4.

2, ⅜″ fish-plates, 6.15 sq. ft. each × 15.3 # =	188.2 #
14, 1″ × 15″ bolts × 4.27 # each =	59.8 #
Total steel =	248.1 #

Splice in Y 8 at middle of span.

2, ⅜″ fish-plates, 4.26 sq. ft. × 15.3 # =	130.4 #
12, 1″ × 15″ bolts × 4.27 # each =	51.2 #
Total steel =	181.6 #

Splice in X 4.

4, 1″ × 11″ bolts × 3.39 # each =	13.6 #

The C. L. weights are computed, doubled, excepting for the middle member 9 9′, and entered on the preliminary weight sheet. The additional weights of the connections are next doubled, except for ends of 9 9′, and entered on the same sheet in a separate column for each material.

681. Preliminary Weight Sheet.—

Member.	*C*-length.	Dimensions.	Weight per foot.	C. L. Weight. Wood.	C. L. Weight. Steel.	Conn. Wood.	Conn. Steel.	Conn. C. Iron.
X 1	10.77	10×10″	25.00	538.6	....	−90.0	....	....
X 3	10.77	10×10″	25.00	538.6	....	0.0	....	....
X 5	10.77	10×10″	25.00	538.6	....	....	+ 27.2	....
X 7	10.77	10×10″	25.00	538.6	....	....	....	....
X 9	10.77	10×10″	25.00	538.6	....	....	....	....
Y 1	10.00	10×12″	30.00	600.0	....	+60.0	+ 513.2	....
Y 2	10.00	10×12″	30.00	600.0	....	....	....	....
Y 4	10.00	10×12″	30.00	600.0	....	....	+ 496.2	....
Y 6	10.00	10×12″	30.00	600.0	....	....	....	....
Y 8	10.00	10×12″	30.00	600.0	....	....	+ 181.6	....
1 2	4.00	1, ½″ R.	0.67	....	+ 5.4	....	+ 2.3	+ 2.8
3 4	8.00	1, ½″ R.	0.67	....	10.8	....	+ 4.4	+ 6.9
5 6	12.00	1, $\frac{11}{16}$″ R.	1.26	....	30.2	....	+ 8.7	+ 16.8
7 8	16.00	1, $\frac{13}{16}$″ R.	1.76	....	56.4	....	+ 12.6	+ 25.6
9 9′	20.00	1,1$\frac{5}{16}$″ R.	4.60	....	92.0	....	+ 22.5	+ 55.2
2 3	10.77	6×6″	9.00	193.8	....	−44.0	....	....
4 5	12.81	6×6″	9.00	230.6	....	28.2	....	....
6 7	15.62	6×8″	12.00	374.8	....	32.8	....	....
8 9	18.87	8×8″	16.00	603.8	....	40.6	....	....
				+7096.0	+194.8	−145.6	+1268.7	+107.3

682. Results of Preliminary Weight Sheet.—

C. L. Weights.		Connection Weights.	
Wood	7096.0	Steel	1268.7
Steel	194.8	C. Iron	107.4
C. L. Weights	7290.8		1376.1
Conn. Weights	1230.5	Deduct wood	145.6
Total weight	8521.3 lbs. = weight of truss.		1230.5 lbs.

Then $\dfrac{8521.3}{15 \times 100} = 5.861$ lbs. per horizontal sq. ft. instead of 5.61 lbs. assumed by application of the formula for weight of truss.

$P = 161.55\ (2 + 3 + 3 + 3 + 5.681 \cos 21.8°) = 2631$ lbs. $= 1.316$ tons per apex.

Hence the P stresses are to be increased in the proportion of $1.316 : 1.310 :: P$ stress : revised P stress, or by $\frac{46}{100}$ of 1 per cent.

These revised P stresses are very quickly computed by a good slide rule.

683. Revised Stress, Dimension, and Weight Sheet.—

Member.	Rev. P-stress.	S-stress.	W-stress.	Maximum.	C-length.	Dimensions.	Weights.
X 1	−16.7	−16.6	− 9.0	−33.3	10.77	As before	As before
X 3	−14.4	−14.3	− 7.8	−28.7	10.77	As before	As before
X 5	−12.7	−12.6	− 6.5	−25.3	10.77	As before	As before
X 7	−10.9	−10.8	− 5.3	−21.7	10.77	As before	As before
X 9	− 9.0	− 9.0	− 4.1	−18.0	10.77	As before	As before
Y 1	+15.0	+14.9	+ 9.7	+29.9	10.00	As before	As before
Y 2	+15.0	+14.9	+ 9.7	+29.9	10.00	As before	As before
Y 4	+13.4	+13.3	+ 8.1	+26.7	10.00	As before	As before
Y 6	+11.8	+11.7	+ 6.6	+23.5	10.00	As before	As before
Y 8	+10.1	+10.0	+ 5.0	+20.1	10.00	As before	As before
1 2	+ 0.0	+ 0.0	+ 0.0	+ 0.0	4.00	As before	As before
3 4	+ 0.7	+ 0.7	+ 0.6	+ 1.4	8.00	As before	As before
5 6	+ 1.3	+ 1.3	+ 1.3	+ 2.6	12.00	As before	As before
7 8	+ 2.0	+ 2.0	+ 1.9	+ 4.0	16.00	As before	As before
9 9′	+ 5.3	+ 5.3	+ 2.6	+10.6	20.00	As before	As before
2 3	− 1.7	− 1.7	− 1.7	− 3.4	10.77	As before	As before
4 5	− 2.1	− 2.1	− 2.0	− 4.2	12.81	As before	As before
6 7	− 2.6	− 2.6	− 2.4	− 5.2	15.62	As before	As before
8 9	− 3.1	− 3.1	− 3.0	− 6.2	18.87	As before	As before

684. Results of Revised Weight Sheet.—The dimensions required for the maximum stresses are determined in the manner previously employed, but the increase in stresses is so small that no changes in dimensions are required. Hence the two weight sheets will coincide and the weights previously obtained may be taken as the actual weights of the members and connections.

The following data are then easily computed and are valuable for comparison with weights of similar trusses in future.

$\frac{7096.0 - 145.6}{8521.3} = 81\frac{57}{100}$ per cent, weight of wood.

$\frac{194.8 + 1268.7}{8521.3} = 17\frac{17}{100}$ per cent, weight of steel.

$\frac{107.3}{8521.3} = 1\frac{26}{100}$ per cent, weight of cast-iron.

$\frac{1230.5}{7290.8} = 16\frac{88}{100}$ per cent to be added to centre length weights to allow for weight of connections at apexes.

EXAMPLE 3.—A STEEL TRUSS

685. A Steel Truss for the Same Roof.— Resume Example 3 of Chapter XI, Art. 571, and of Chapter XII, Art. 637, which is a steel truss for the same purpose as the wooden truss in Example 2. Each member is composed of two angles, the connections at apexes being made by riveted gussets placed between them and at splices by riveted cover plate over them.

Resume dimension sheet of Art. 593, Chapter XI, and details of connections as in Art. 601, Chapter XII. Both legs of the angles are riveted, and $\frac{3}{4}''$ rivets are used throughout. Gussets are made $\frac{5}{8}''$ thick to develop full shear in rivets.

686. Centre Length Weights.—These are easily computed from the data on the preliminary weight sheet, doubling the weight of each member, except for the middle vertical, then inserting in the proper column.

687. Weights of Connections.—Computed separately for each apex in accordance with details in Figs. 601 to 606. These are also doubled, except for the apexes at ends of the middle vertical, and are then inserted in the weight sheet.

Connection of X 1 and Y 1 (Figs. 601, 602, 603).

Extra length of X 1, $0.52' \times 22.6$ # =	12.0
2 pieces, $5 \times 3 \times \frac{1}{2}$ Ls, $1.05' \times 25.6$ # =	26.9
2 pieces, $3 \times 3 \times \frac{1}{2}$ Ls, $1.05' \times 16.6$ # =	17.4
2 pieces, $6 \times 6 \times 1$ Ls, $1.00' \times 74.8$ # =	74.8
Gusset, 2.29 sq. ft. $\times$ 25.5 # =	58.4
98 rivet heads $\times \frac{1}{6}$ # =	16.3
	205.8
Deduct for Y 1, $0.65' \times 19.6$ # =	12.7
	$193.1 \times 2 = 386.2$ #

Connection of X 1, X 3, etc.

2 pieces, 2½ × 2½ × ¼ Ls, 0.45′ × 8.2 # = ...		3.7	
Gusset, 0.78 sq. ft. × 25.5 # = ...		19.9	
40 rivet heads × 1/6 # = ...		6.7	
		30.3	
Deduct for 1 2, 0.27′ × 5.0 # = ...	1.4		
Deduct for 2 3, 0.46′ × 9.8 # = ...	4.5	5.9	
		24.4 × 2 =	48.8 #

Connection of Y 1, Y 2, etc.

Gusset, 0.15 sq. ft. × 25.5 # = ...		3.8	
4 rivet heads × 1/6 # = ...		0.7	
		4.5	
Deduct for 1 2, 0.25′ × 5.0 # = ...		1.3	
		3.2 × 2 =	6.4 #

Connection of X 3, X 5, etc. Splice.

2 pieces, 3 × 2½ × ¼ Ls, 0.50′ × 9.0 # = ...		4.5	
2 pieces, 2 × 2 × 5/16 Ls, 0.55′ × 5.0 # = ...		2.8	
Gusset, 1.00 sq. ft. × 25.5 # = ...		25.5	
Cover, 1.10′ × 6⅝ × ⅜, 1.10′ × 8.45 # = ...		9.3	
64 rivet heads × 1/6 # = ...		10.7	
Deduct for 3 4, 0.27′ × 5.0 # = ...	1.4	52.8	
Deduct for 4 5, 0.35′ × 14.4 # = ...	5.0	6.4	
		46.4 × 2 =	92.8 #

Connection of Y 2, Y 4, etc. Splice.

2 pieces, 2½ × 2 × ¼ Ls, 0.70′ × 5.6 # = ...		3.9	
2 pieces, 2 × 2 × 5/16 Ls, 0.55′ × 5.0 # = ...		2.8	
Gusset, 1.12 sq. ft. × 25.5 # = ...		28.6	
Cover, 1.08′ × 6⅝ # × 5/16″, 1.08′ × 7.04 # = ...		7.6	
66 rivet heads × 1/6 # = ...		11.0	
		53.9	
Deduct for 2 3, 0.60′ × 9.8 # = ...	5.9		
Deduct for 3 4, 0.25′ × 5.0 # = ...	1.3	7.2	
		46.7 × 2 =	93.4 #

Connection of X 5, X 7, etc.

2 pieces, 2 × 2 × $\frac{3}{16}$ Ls, 0.50′ × 5.0 # = ...		2.5
2 pieces, $3\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{1}{4}$ Ls, 0.65′ × 9.8 # = ...		6.4
Gusset, 1.12 sq. ft. × 25.5 # = ...		28.6
40 rivet heads × 1/6 # = ...		6.7
		44.2
Deduct for 5 6, 0.30′ × 5.0 # = ...	1.5	
Deduct for 6 7, 0.30′ × 17.4 # = ...	5.2	6.7
		37.5×2 = 75.0 #

Connection of Y 4, Y 6, etc.

2 pieces, 2 × 2 × $\frac{3}{16}$ Ls, 0.60′ × 5.0 # = ...		3.0
2 pieces, 3 × $2\frac{1}{2}$ × $\frac{1}{4}$ Ls, 0.70′ × 9.0 # = ...		6.3
Gusset, 0.87 sq. ft. × 25.5 # = ...		29.2
48 rivet heads × 1/6 # = ...		8.0
		39.5
Deduct for 4 5, 0.30′ × 14.4 # = ...	4.3	
Deduct for 5 6, 0.25′ × 5.0 # = ...	1.3	5.6
		33.9×2 = 67.8 #

Connection of X 7, X 9, etc. Splice.

2 pieces, 2 × 2 × $\frac{3}{16}$ Ls, 0.60′ × 5.0 # = ...		3.0
2 pieces, 4 × 3 × $\frac{5}{16}$ Ls, 0.60′ × 14.4 # = ...		8.6
Gusset, 1.42 sq. ft. × 25.5 # = ...		36.2
Cover, 1.12′ × $6\frac{5}{8}$ × $\frac{5}{16}$″, 1.12′ × 7.24 # = ...		7.9
66 rivet heads × 1/6 # = ...		11.0
		66.7
Deduct for 7 8, 2 × 2 × $\frac{3}{16}$, 0.25′ × 5.0 # =	1.3	
Deduct for 8 9, 4 × 3 × $\frac{5}{16}$, 0.25′ × 14.4 # =	3.6	4.9
		61.8×2 = 123.6 #

Connection of Y 6, Y 8, etc. Splice.

2 pieces, $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$ Ls, $0.60' \times 9.8$ # = ...		5.8	
2 pieces, $2 \times 2 \times \frac{3}{16}$ Ls, $0.60' \times 5.0$ # = ...		3.0	
Gusset, 1.02 sq. ft. $\times$ 25.5 # = ...		26.0	
Cover, 1.12×7.04 # = ...		7.9	
70 rivet heads $\times$ 1/6 # = ...		11.7	
		54.4	
Deduct for 6 7, $0.35' \times 17.4$ # = ...	6.1		
Deduct for 7 8, $0.25' \times 5.0$ # = ...	1.3	7.4	
		$47.0 \times 2 =$	94.0 #

Connection of X 9, X 9′, etc. Splice.

2 pieces, $2\frac{1}{2} \times 2 \times \frac{1}{4}$ Ls, $0.55' \times 7.4$ # = ...		4.1	
Gusset, 0.77 sq. ft. $\times$ 25.5 # = ...		19.7	
2 covers, $2.0' \times 0.45'$, 0.9 sq. ft. $\times$ 10.2 # = ...		9.2	
Cover, $1.0' \times 6\frac{5}{8} \times \frac{1}{4}''$, $1.0' \times 6.0$ # = ...		6.0	
48 rivet heads $\times$ 1/6 # = ...		8.0	
		47.0	
Deduct for 9 9′, $0.40' \times 8.2$ # = ...		3.3	
		$43.7 \times 1 =$	43.7 #

Connection of Y 8, Y 8′, etc.

2 pieces, $2\frac{1}{2} \times 2 \times \frac{1}{4}$ Ls, $0.85' \times 7.4$ # = ...		6.3	
2 pieces, $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$ Ls, $0.70' \times 9.8$ # = ...		6.9	
2 pieces, $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$ Ls, $0.70' \times 9.8$ # = ...		6.9	
Gusset, 2.18 sq. ft. $\times$ 25.5 # = ...		55.6	
88 rivet heads $\times$ 1/6 # = ...		14.7	
		90.4	
Deduct for 8 9, $0.4' \times 24.6$ # = ...	9.9		
Deduct for 8 9″, $0.4' \times 24.6$ # = ...	9.9		
Deduct for 9 9″, $0.25' \times 8.2$ # = ...	2.1	21.9	
		$68.5 \times 1 =$	68.5 #

688. Preliminary Weight Sheet.—

Member.	C-length.	Dimensions.	Weight per foot.	C. L. Weight.	Conn. Weight.
X 1	10.77′	2, 5×3×$\frac{7}{16}$ Ls	22.6 #	486.8	+386.2
X 3	10.77	2, 5×3×$\frac{7}{16}$	22.6	486.8	48.8
X 5	10.77	2, 5×3×$\frac{3}{8}$	19.6	422.2	92.8
X 7	10.77	2, 5×3×$\frac{3}{8}$	19.6	422.2	75.0
X 9	10.77	2, 5×3×$\frac{5}{16}$	16.4	353.3	123.6
Y 1	10.00	2, 5×3×$\frac{3}{8}$	19.6	392.0	6.4
Y 2	10.00	2, 5×3×$\frac{3}{8}$	19.6	392.0	93.4
Y 4	10.00	2, 5×3×$\frac{3}{8}$	19.6	392.0	67.8
Y 6	10.00	2, 5×3×$\frac{3}{8}$	19.6	392.0	94.0
Y 8	10.00	2, 5×3×$\frac{5}{16}$	16.4	328.0	68.5
1 2	4.00	2, 2×2×$\frac{3}{16}$	5.0	40.0	....
3 4	8.00	2, 2×2×$\frac{3}{16}$	5.0	80.0	....
5 6	12.00	2, 2×2×$\frac{3}{16}$	5.0	120.0	....
7 8	16.00	2, 2×2×$\frac{3}{16}$	5.0	160.0	....
9 9′	20.00	2, 2½×2½×¼	8.2	164.0	43.7
2 3	10.77	2, 3½×2½×¼	9.8	211.1	....
4 5	12.81	2, 4×3×$\frac{5}{16}$	14.4	368.9	....
6 7	15.62	2, 5×3½×$\frac{5}{16}$	17.4	543.6	....
8 9	18.87	2, 6×4×$\frac{3}{8}$	24.6	928.4	....
				6683.3	1100.2

689. Results Obtained from Preliminary Weight Sheet.—

C.L. weights......................	6683.3
Conn. weights.....................	1100.2
Total..........................	7783.8 lbs.

Then $\frac{7783.8}{1500.0}$ = 5.189 lbs. per horizontal sq. ft. instead of 5.161 lbs. assumed by the formula.

P = 161.55 (2 + 3 + 3 + 3 + 5.189 cos 21.8°) = 2555 lbs. = 1.278 tons.

Hence 1.149 : 1.278 : : P stress : revised P stress.

Entering the revised P stresses, S stresses, and W stresses on the Revised Weight Sheet and computing the maximum stress, the sectional dimensions of the members are determined as before. No changes are found to be necessary in these dimensions, although the P stresses have been slightly increased. Therefore the preliminary total weight may be taken as the revised total weight of the truss.

690. Revised Weight Sheet.—

Member.	Rev. *P*-stress.	*S*-stress.	Rev. Maximum.	*C*-length.	Dimensions.
X 1	−16.1	−16.6	−32.7	10.77	As before
X 3	−13.9	−14.3	−28.2	10.77	As before
X 5	−12.2	−12.6	−24.8	10.77	As before
X 7	−10.5	−10.8	−21.3	10.77	As before
X 9	− 8.8	− 9.0	−17.8	10.77	As before
Y 1	+14.5	+14.9	+29.4	10.00	As before
Y 2	+14.5	+14.9	+29.4	10.00	As before
Y 4	+13.0	+13.3	+26.3	10.00	As before
Y 6	+11.4	+11.7	+23.1	10.00	As before
Y 8	+ 9.7	+10.0	+19.7	10.00	As before
1 2	+ 0.0	+ 0.0	+ 0.0	4.00	As before
3 4	+ 0.7	+ 0.7	+ 1.4	8.00	As before
5 6	+ 1.2	+ 1.3	+ 2.5	12.00	As before
7 8	+ 1.9	+ 2.0	+ 3.9	16.00	As before
9 9′	+ 5.1	+ 5.2	+10.3	20.00	As before
2 3	− 1.6	− 1.7	− 3.3	10.77	As before
4 5	− 2.0	− 2.1	− 4.1	12.81	As before
6 7	− 2.5	− 2.6	− 5.1	15.62	As before
8 9	− 3.1	− 3.1	− 6.2	18.87	As before

691. Final Results.—

Then $\frac{1100.2}{6683.3} = 16\frac{46}{100}$ per cent to be added to C. L. weights to allow for weight of connections at apexes.

EXAMPLES 5 A AND B.—STEEL FINK TRUSS WITH CAMBERED LOWER CHORD

692. General Description.—Resume Example 5, Art. 118, Fig. 76, of Chapter IV, Chapter XI, and Chapter XII. This is a steel truss of 128 ft. span, divided into 16 equal panels, with lower chord cambered 2 ft. at the middle.

Under present conditions, it would not be economical to construct a Fink truss of wooden timbers and steel rods on account of cost of timber and the difficulty of making connections at the apexes. The wooden truss would also probably cost more than one entirely of steel.

In making this truss of steel, either one of two general systems of construction may be used for the sections of the members and their connections at the apexes.

1. Upper chord is composed of pairs of channels latticed together on top and bottom flanges; each web strut consists of a pair of angles riveted to an eye-gusset at each end; each web tie and each member of the lower chord is composed of a pair of loop-welded eye-rods, the eye-ends being flattened to 4/5 diameter; all connections are made on pins with each end recessed and with pin nuts.

2. Each member of the truss consists of a pair of angles; all connections are riveted, made with gussets, and covers at splices, if necessary.

Both systems will be applied to this example in order to determine their comparative economy in weight and cost.

A.—Pin Connections

693. Description.—Resume dimension sheet of Art. 593, Chapter XI. The *P*, *S*, and maximum stresses, *C* lengths, and weight per foot are entered on Preliminary Weight Sheet; the weights of connections are then computed from details and data for same example in Chapter XII and also entered.

694. Weight of Latticing for One Member of Upper Chord.—To make the member equally stiff both horizontally and vertically, the channels are set 6″ apart and latticed together with single bars, ¼ × 2″, using ¾″ rivets.

4 batten plates, 0.90′ 5½ × ¼″; 3.6 ft. × 4.68 # = ...	16.8
78 lattice bars, 1.04′ × 2 × ¼″, 81.1 ft. × 1.70 # = ...	137.9
80 rivet heads × 1/6 # = ...	13.3
Total...	168.0

For both sides of roof, 168.0 × 2 = 336.0 #

695. Weights of Connections at Apexes.—Connection of *X* 1 and *Y* 1, omitting expansion rolls and seats (Figs. 607, 608, 609).

2, 9″ 15 # channels, 0.40′ × 30 # = ...	12.0
2 plates, 1.40′ × 8 × ½″, 1.40′ × 2 × 13.6 # = ...	38.1
40 rivet heads × 1/6 # = ...	6.7
$3\frac{5}{16}$″ pin, body, 24.4 #, 2 ends, 4.2 #, 2 nuts, 6.0 # =	34.6
2 loop ends on 2½″ rods, 3.83′ × 16.7 # = ...	64.4
Total..................................	155.4×2=310.8 #

Connection of X 1, X 2; X 5, X 6; X 9, X 10; X 13, X 14, etc. (Figs. 607, 615).

$1\frac{9}{16}''$ pin; body, 3.7 # ; ends, 1.5 # ; nuts, 5.0 # = ..	10.2
For 1 2, $0.67' \times 2 \times \frac{5}{8}''$ = 0.67×4.25 # = ...	2.9
4 rivet heads × 1/6 # = ...	0.7
	13.8×2 = 27.6 #

Connection of Y 1, Y 3, etc. (Fig. 607).

$3\frac{7}{16}''$ pin; body, 25.2 # ; ends, 4.2 # ; nuts, 6.0 # = .	35.4
Y 1, 2 loop ends on $2\frac{1}{2}''$ rods, $4.0' \times 16.7$ # = ...	66.8
Y 3, 2 loop ends on $2\frac{3}{8}''$ rods, $3.8' \times 15.1$ # = ...	57.4
2 3, 2 loop ends on $\frac{11}{16}''$ rods, $2.8' \times 1.3$ # = ...	3.6
1 2, $0.67'$, $2 \times \frac{5}{8}''$ = 0 # 67 × 4.25 # = ...	2.9
4 rivet heads × 1/6 # = ...	0.7
	166.8×2 = 333.6 #

Connection of X 2, X 5; X 10, X 13, etc. (Figs. 607, 609, 618).

$1\frac{13}{16}''$ pin; body, 5.3 # ; ends, 1.5 # ; nuts, 5.0 # = ...	11.8
3 4, $0.75'$, $2 \times \frac{5}{8}''$ = 0.75×4.25 # = ...	3.2
4 rivet heads × 1/6 # = ...	0.7
2 3, 2 loop ends on $\frac{11}{16}''$ rods = $1.7' \times 1.26$ # = ...	1.1
4 5, 2 loop ends on $\frac{11}{16}''$ rods = $1.7' \times 1.26$ # = ...	1.1
	17.9×2 = 35.8 #

Connection of Y 3, Y 7, etc. (Figs. 607, 618).

$3\frac{7}{16}''$ pin; body, 25.2 # ; ends, 4.2 # ; nuts, 6.0 # = .	35.4
Y 3, 2 loop ends on $2\frac{3}{8}''$ rods, $4.0' \times 16.7$ # = ...	66.8
Y 7, 2 loop ends on $2\frac{1}{4}''$ rods, $3.8' \times 13.5$ # = ...	51.3
4 7, 2 loop ends on $1''$ rod, $3.0' \times 2.67$ # = ...	8.0
3 4, $0.75' \times 2 \times \frac{5}{8}''$ = $0.75' \times 4.25$ # = ...	3.2
4 rivet heads × 1/6 # = ...	0.7
	165.4×2 = 330.8 #

Connection of X 6, X 9, etc. (Figs. 607, 617, 619).

$1\frac{7}{8}''$ pin; body, 5.9 # ; ends, 1.5 # ; nuts, 5.0 # = ...	12.4
2 plates, $2 \times 2.75' \times 8 \times \frac{1}{2}$ = $2.75 \times 2 \times 13.6$ = .	74.8
7 8, $1.08' \times 4 \times \frac{5}{8}''$ = $1.08' \times 8.5$ # = ...	9.2
6 7, 2 loop ends on rods, $2.2' \times 3.79$ # = ...	8.3
8 9, 2 loop ends on rods, $2.2' \times 3.79$ # = ...	8.3
108 rivet heads × 1/6 # = ...	19.0
	131.0×2 = 262.0 #

Connection of *Y* 7, *Y* 15, etc. (Figs. 607, 619).

$2\frac{11}{16}''$ pin; body, 15.1 # ; ends, 2.0 # ; nuts, 5.0 # = .	22.1
Y 7, 2 loop ends, 3.3′ × 13.5 # = ...	44.5
Y 15, 2 loop ends, 3.0′ × 8.8 # = ...	26.4
8 15, 2 loop ends, 2.7′ × 4.60 # = ...	12.4
7 8, 1.08′ × 4 × $\frac{5}{8}''$ = 1.08 × 8.5 # = ...	9.2
4 rivet heads × 1/6 # = ...	0.7
	115.3×2=230.6 #

Connection of 4 7, 6 7; 8 9, 8 11, etc. (Fig. 607).

$1\frac{11}{16}''$ pin; body, 3.5 # ; ends, 1.5 # ; nuts, 5.0 # = . .	10.0
4 7, 2 loop ends on rods, 2.0′ × 2.67 # = ...	5.4
6 7, 2 loop ends on rods, 2.0′ × 3.77 # = ...	7.5
4 5, 2 loop ends on rods, 1.7′ × 1.26 # = ...	2.1
5 6, 0.67′ × 2 × $\frac{5}{8}''$ = 0.67 × 5.25 # = ...	3.7
4 rivet heads × 1/6 # = ...	0.7
	29.4×2= 58.8 #

Connection of 8 15, 12 15, etc. (Fig. 607).

$1\frac{15}{16}''$ pin; body, 5.9 # ; ends, 1.5 # ; nuts, 5.0 # = . .	12.4
8 15, 2 loop ends on rods, 2.2′ × 4.6 # = ...	10.1
12 15, 2 loop ends on rods, 2.4′ × 7.05 # = ...	16.9
8 11, 2 loop ends on rods, 2.0′ × 2.7 # = ...	5.4
11 12, 0.75′ × 4.25 # = ...	3.2
4 rivet heads × 1/6 # = ...	0.7
	48.7×2= 97.4 #

Connection of 12 15, 14 15, etc. (Fig. 607).

$2\frac{7}{16}''$ pin; body, 9.81 # ; ends, 2.0 # ; nuts, 5.0 # =	16.8
12 15, 2 loop ends on rods, 2.75′ × 7.05 # = ...	19.3
12 13, 2 loop ends on rods, 2.20′ × 1.26 # = ...	2.8
14 15, 2 loop ends on rods, 2.75′ × 8.18 # = ...	22.5
13 14, 0.67′ × 4.25 # = ...	2.8
4 rivet heads × 1/6 # = ...	0.7
	64.9×2=129.8 #

Connection of *X* 14, *X* 14′, etc. (Figs. 607, 620).

$3\frac{3}{16}''$ pin; body, 17.5 # ; ends, 2.5 # ; nuts, 6.0 # =	26.0
2 plates, 2.42′ × 8 × $\frac{5}{16}''$ = 2.42 × 8.5 = ...	20.6
14 15, 2 loop ends, 3.25′ × 16.4 # = ...	53.3
14′ 15′, 2 loop ends, 3.25′ × 16.4 # = ...	53.3
15 15′, 1 loop end, 1.67′ × 1.3 # = ...	2.2
80 rivet heads × 1/6 # = ...	13.3
	168.7×1=168.7 #

Connection of *Y* 15, *Y* 15′, etc. (Fig. 607).

$2\frac{11}{16}''$ pin; body, 10.6 #; ends, 2.5 #; nuts, 6.0 # = 19.1
Y 14, 2 loop ends, 3.0′ × 8.77 # = ... 24.3
Y 14′, 2 loop ends, 3.0′ × 8.77 # = ... 24.3
15 15′, 1 loop end, 1.17′ × 1.3 # = ... 1.5

69.2×1 = 69.2 #

696. Preliminary Weight Sheet.—

Member.	*C*-length.	Dimensions.	Weight per foot.	C. L. Weights.	Lattices.	Connections.
X 1	8.61	2, 9″×15 # Ch.	30.0	516.6	336.0	310.8
X 2	8.61	2, 9″×15 #	30.0	516.6	336.0	27.6
X 5	8.61	2, 9″×15 #	30.0	516.6	336.0	34.8
X 6	8.61	2, 9″×15 #	30.0	516.6	336.0	27.6
X 9	8.61	2, 9″×15 #	30.0	516.6	336.0	262.0
X 10	8.61	2, 9″×15 #	30.0	516.6	336.0	27.6
X 13	8.61	2, 9″×15 #	30.0	516.6	336.0	34.8
X 14	8.61	2, 9″×15 #	30.0	516.6	336.0	27.6
Y 1	9.17	2, $2\frac{1}{2}''$ R.	33.4	612.6	....	333.6
Y 3	9.17	2, $2\frac{3}{8}''$ R.	30.1	552.0	....	330.8
Y 7	18.34	2, $2\frac{1}{4}''$ R.	27.0	1458.0	....	230.6
Y 15	27.36	2, $1\frac{13}{16}''$ R.	17.5	957.6	....	69.2
1 2	3.14	2, $2\times2\times\frac{3}{16}''$ Ls	5.0	31.4	....	....
3 4	6.27	2, $2\frac{1}{2}\times\frac{1}{4}''$	7.4	92.8	....	....
5 6	3.14	2, $2\times2\times\frac{3}{16}''$	5.0	31.4	....	....
7 8	12.54	2, $4\times3\times\frac{5}{16}''$	14.4	361.2	....	....
9 10	3.14	2, $2\times2\times\frac{3}{16}''$	5.0	31.4	....	....
11 12	6.27	2, $2\frac{1}{2}\times2\times\frac{1}{4}''$	7.4	92.8	....	....
13 14	3.14	2, $2\times2\times\frac{3}{16}''$	5.0	31.4	....	....
2 3	9.17	2, $\frac{11}{16}''$ R.	2.5	45.9	....	....
4 5	9.17	2, $\frac{11}{16}''$ R.	2.5	45.9	....	....
4 7	9.17	2, 1″ R.	5.3	97.2	....	....
6 7	9.17	2, $1\frac{3}{16}''$ R.	7.5	137.6	....	....
8 9	9.17	2, $1\frac{3}{16}''$ R.	7.5	137.6	....	58.8
8 11	9.17	2, 1″ R.	5.3	97.2	....	....
8 15	18.34	2, $1\frac{5}{16}''$ R.	9.2	337.5	....	97.4
10 11	9.17	2, $\frac{11}{16}''$ R.	2.5	45.9	....	....
12 13	9.17	2, $\frac{11}{16}''$ R.	2.5	45.9	....	....
12 15	9.17	2, $1\frac{5}{8}''$ R.	14.1	258.6	....	129.8
14 15	9.17	2, $1\frac{3}{4}''$ R.	16.4	300.8	....	....
15 15′	23.60	1, $\frac{11}{16}''$ R.	1.3	30.7	....	168.7
		Totals.........	....	9966.2	2688.0	2230.5

697. Results of Preliminary Weight Sheet.—

C. L. weights = ...	9966.2
Latticing upper chord = ...	2688.0
Connections at apexes = ...	2230.5
Total weight of truss	14884.7 lbs.

Then $\dfrac{14884.7}{16 \times 128} = 7.268$ lbs. per horizontal sq. ft. instead of 6.43 lbs. assumed by formula.

$P = 137.92\ (2 + 3 + 3 + 3 + 7.268 \cos 21.8°) = 2448$ lbs. = 1.224 tons apex load instead of 1.171 tons as previously computed.

Then $\dfrac{1.224 - 1.171}{1.171} = 4\dfrac{44}{100}$ per cent, increase in P stresses.

Computing these increases and adding them to the corresponding P stresses, obtaining the maximum stresses in members, then revising their sectional dimensions as before, we find that no changes in these are required. Therefore the dimensions and weights on the preliminary weight sheet may be taken as the actual weights of the truss.

If the latticing of the members of the upper chord be added to the C. L. weights, we then obtain:

C. L. weights = ...	9966.2
Latticing = ...	2688.0
Total	12654.2

Then $\dfrac{2230.5}{12654.2} = 17\dfrac{63}{100}$ per cent to be added to C. L. weights for the weights of connections at apexes.

B.—RIVETED CONNECTIONS

698. Description.—Same as Example 5 A, but entirely constructed of angles with gussets and riveted connections at all apexes. Resume dimension sheet, Art. 599, Chapter XI, and details of connections in Arts. 655, 656, of Chapter XII.

699. Weights of Connections at Apexes.—Connection of $X\,1$ and $Y\,1$ (Figs. 621, 622).

X 1, add 1.00′ × 36.2 # =		36.2
2 pieces, 6 × 4 × 3/8″, 1.8′ × 24.6 # =		44.3
2 pieces, 3½ × 3½ × 3/8″, 1.9′ × 17.5 # =		32.3
2 pieces, 8 × 8 × 1⅛″, 1.75′ × 113.8 # =		199.2
Gusset, 3.37 sq. ft. × 25.5 # =		85.9
132 rivet heads × 1/6 # =		22.0
		419.9
Y 1, deduct 1.25′ × 34.2 # =		42.8
		377.1×2=754.2 #

Connection of *X* 1, *X* 2; *X* 5, *X* 6; *X* 9, *X* 10; *X* 13, *X* 14, etc. (Figs. 621, 624).

Gusset bar, 2 × 5/8, 1.00′ × 4.25 # =		4.3	
8 rivet heads × 1/6 # =		1.3	
		5.6	
1 2, deduct 0.5′ × 5.0 # =		2.5	
		3.1×2=	6.2 #

Connection of *Y* 1, *Y* 3, etc.

2 pieces, 2 × 2 × 3/16″, 0.75 × 5.0 # =		3.8	
Gusset, 0.78 sq. ft. × 25.5 # =		19.9	
30 rivet heads × 1/6 # =		5.0	
		28.7	
1 2, deduct 0.55′ × 5.0 # =	2.8		
2 3, deduct 0.90′ × 5.0 # =	4.5	7.3	
		21.4×2=	42.8 #

Connection of *X* 2, *X* 5; *X* 10, *X* 13, etc. (Fig. 621).

2 pieces, 2 × 2 × 3/16″, 0.75′ × 5.0 # =		3.5	
2 pieces, 2 × 2 × 3/16″, 0.75′ × 5.0 # =		3.5	
2 pieces, 2 × 2 × 3/16″, 0.75′ × 5.0 # =		3.5	
Gusset, 1.74 sq. ft. × 25.5 =		44.6	
60 rivet heads × 1/6 # =		10.0	
		65.1	
2 3, deduct 1.2′ × 5.0 # =	6.0		
3 4, deduct 0.45′ × 5.6 # =	2.5		
4 5, deduct 1.20′ × 5.0 # =	6.0		
	14.5	14.5	
		50.6×2=101.2 #	

Connection of Y 3, Y 7, etc. Splice (Fig. 621).

Cover $7\frac{5}{8} \times \frac{3}{8}$, 2.75′ × 9.2 # = ...		25.3
Web covers, $5\frac{1}{2} \times \frac{7}{16}$, 2.8′ × 8.2 # = ...		23.0
2 pieces, 2 × 2 × $\frac{3}{16}$, 0.75′ × 5.0 # = ...		3.8
2 pieces, 2 × 2 × $\frac{1}{4}$, 0.75′ × 5.6 # = ...		4.2
Gusset, 1.08 sq. ft. × 25.5 # = ...		27.6
82 rivet heads × 1/6 # = ...		13.7
		97.6
3 4, deduct 0.5′ × 5.6 # = ...	2.8	
4 7, deduct 0.9′ 5.6 # = ...	5.0	
	7.8	7.8
		89.8×2 = 179.6 #

Connection of X 6, X 9, etc. Splice (Figs. 621, 623).

Cover $8\frac{5}{8} \times \frac{3}{8}$, 2.8′ × 11.0 # = ...		30.8
Web covers, $5\frac{1}{2} \times \frac{3}{8}$, 2.8′ × 7.0 # = ...		19.6
2 pieces, $2\frac{1}{2}$ × 2 × $\frac{1}{4}$, 0.8′ × 7.4 # = ...		5.9
2 pieces, $2\frac{1}{2}$ × 2 × $\frac{1}{4}$, 0.8′ × 7.4 # = ...		5.9
2 pieces, 3 × $2\frac{1}{2}$ × $\frac{5}{16}$, 0.8′ × 11.2 # = ...		9.0
Gusset, 2.16 sq. ft. × 25.5 # = ...		55.0
98 rivet heads × 1/6′ # = ...		16.3
		142.5
6 7, deduct 1.2′ × 8.2 # = ...	9.9	
8 9, deduct 1.2′ × 8.2 # = ...	9.9	
7 8, deduct 0.45′ × 14.4 # = ...	6.5	
	26.3	26.3
		116.2×2 = 232.4 #

Connection of Y 7, Y 15, etc. Splice (Fig. 621).

Cover $7\frac{5}{8} \times \frac{3}{8}$, 2.0′ × 9.7 # = ...		19.4
2 pieces, 3 × $2\frac{1}{2}$ × $\frac{5}{16}$, 1.1′ × 14.4 # = ...		15.9
2 pieces, $2\frac{1}{2}$ × 2 × $\frac{5}{16}$, 1.0′ × 9.0 # = ...		9.0
Gusset, 1.33 sq. ft. × 25.5 # = ...		33.9
78 rivet heads × 1/6 # = ...		13.0
		91.2
7 8, deduct 0.4′ × 14.4 # = ...	5.8	
8 15, deduct 0.8′ × 9.8 # = ...	7.8	
	13.6	13.6
		77.6×2 = 155.2 #

Connection of 4 7, 6 7; 8 9, 8 11, etc. (Fig. 621).

2 pieces, $2 \times 2 \times \frac{3}{16}$, $0.75' \times 5.0$ # = ...		3.8
Gusset, 0.49 sq. ft. $\times$ 25.5 # = ...		12.5
24 rivet heads $\times$ 1/6 # = ...		4.0
		20.3
4 5, deduct $0.35' \times 5.0$ # = ...	1.8	
5 6, deduct $0.25' \times 5.0$ # = ...	1.3	
	3.1	3.1
		$17.2 \times 2 =$ 34.4 #

Connection of 8 15, 12 15, etc. Splice (Fig. 621).

Cover $5\frac{5}{8} \times \frac{3}{8}$, $2.05' \times 7.2$ # = ...		14.8
2 pieces, $2 \times 2 \times \frac{1}{4}$, $0.75' \times 6.4$ # = ...		4.8
2 pieces, $2 \times 2 \times \frac{3}{16}$, $0.90' \times 5.0$ # = ...		4.5
Gusset, 0.79 sq. ft. $\times$ 25.5 # = ...		20.1
62 rivet heads $\times$ 1/6 # = ...		10.3
		54.5
8 11, deduct $0.4' \times 7.4$ # = ...	3.0	
11 12, deduct $0.25' \times 5.6$ # = ...	1.4	
	4.4	4.4
		$50.1 \times 2 = 100.2$ #

Connection of 12 15, 14 15, etc. Splice (Fig. 621).

Cover $5\frac{5}{8} \times \frac{7}{16}''$, $2.05' \times 8.4$ # = ...		17.2
2 pieces, $2 \times 2 \times \frac{3}{16}$, $0.75' \times 5.0$ # = ...		3.8
Gusset, 0.86 sq. ft. $\times$ 25.5 # = ...		21.9
56 rivet heads $\times$ 1/6 # = ...		9.3
		52.2
12 13, deduct $0.45' \times 5.0$ # = ...	2.0	
13 14, deduct $0.25' \times 5.0$ # = ...	1.3	
	3.3	3.3
		$48.9 \times 2 =$ 97.8 #

Connection of X 14, X 14′, etc. Splice (Figs. 621, 626).

Cover $8\frac{5}{8} \times \frac{5}{16}$, 1.41′ × 9.2 # = ...		13.0
2 pieces, $2\frac{1}{2} \times 2 \times \frac{5}{16}$, 0.8′ × 9.0 # = ...		7.2
2 pieces, $2\frac{1}{2} \times 2 \times \frac{5}{16}$, 0.8′ × 9.0 # = ...		7.2
Gusset, 2.39 sq. ft. × 25.5 # = ...		61.0
88 rivet heads × 1/6 # = ...		14.7
		103.1
14 15, deduct 1.25′ × 16.6 # = ...	20.8	
15′ 14′, deduct 1.25′ × 16.6 # = ...	20.8	
15 15′, deduct 0.75′ × 5.0 # = ...	3.8	
	45.4	45.4
		57.7×1 = 57.7 #

Connection of Y 15, Y 15′, etc. Splice (Fig. 621).

Cover $6\frac{5}{8} \times \frac{5}{16}$, 1.8′ × 7.0 # = ...	12.6
Gusset, 0.75 sq. ft. × 25.5 # = ...	19.1
36 rivet heads × 1/6 # = ...	6.0
	37.7
15 15′, deduct 0.35′ × 5.0 # = ...	1.8
	35.9×1 = 35.9 #

700. Preliminary Weight Sheet.—

Member.	C-length.	Dimensions.	Weight per foot.	C. L. Weights.	Connections.
X 1	8.61	2, $6\times4\times\frac{9}{16}''$ Ls.	36.2	623.4	+754.2
X 2	8.61	2, $6\times4\times\frac{9}{16}''$	36.2	623.4	6.2
X 5	8.61	2, $6\times4\times\frac{9}{16}''$	36.2	623.4	6.2
X 6	8.61	2, $6\times4\times\frac{9}{16}''$	36.2	623.4	101.2
X 9	8.61	2, $6\times4\times\frac{1}{2}''$	32.4	557.9	232.4
X 10	8.61	2, $6\times4\times\frac{1}{2}''$	32.4	557.9	6.2
X 13	8.61	2, $6\times4\times\frac{1}{2}''$	32.4	557.9	101.2
X 14	8.61	2, $6\times4\times\frac{1}{2}''$	32.4	557.9	6.2
Y 1	9.17	2, $6\times3\frac{1}{2}\times\frac{9}{16}''$	34.2	627.2	42.8
Y 3	9.17	2, $6\times3\frac{1}{2}\times\frac{9}{16}''$	34.2	627.2	177.6
Y 7	18.34	2, $6\times3\frac{1}{2}\times\frac{7}{16}''$	27.0	990.4	155.2
Y 15	27.36	2, $5\times3\times\frac{7}{16}''$	22.6	1236.7	....
1 2	3.14	2, $2\times2\times\frac{3}{16}''$	5.0	31.4	....
3 4	6.27	2, $2\frac{1}{2}\times2\times\frac{3}{16}''$	5.6	70.2	....
5 6	3.14	2, $2\times2\times\frac{3}{16}''$	5.0	31.4	....
7 8	12.54	2, $4\times3\times\frac{5}{16}''$	14.4	361.2	....

Preliminary Weight Sheet.—Continued.

Member.	*C*-length	Dimensions.	Weight per foot.	C. L. Weights.	Connections.
9 10	3.14	2, 2×2×$\frac{3}{16}''$	5.0	31.4	....
11 12	6.27	2, 2½×2×$\frac{3}{16}''$	5.6	70.2	....
13 14	3.14	2, 2×2×$\frac{3}{16}''$	5.0	31.4	....
2 3	9.17	2, 2×2×$\frac{3}{16}''$	5.0	91.7	....
4 5	9.17	2, 2×2×$\frac{3}{16}''$	5.0	91.7	34.4
4 7	9.17	2, 2½×2×¼″	7.4	135.7	....
6 7	9.17	2, 2½×2½×¼″	8.2	150.4	....
8 9	9.17	2, 2½×2½×¼″	8.2	150.4	....
8 11	9.17	2, 2½×2×¼″	7.4	135.7	34.4
8 15	18.34	2, 3½ 2½ ¼″	9.8	359.5	100.2
10 11	9.17	2, 2×2×$\frac{3}{16}''$	5.0	91.7	....
12 13	9.17	2, 2×2×$\frac{3}{16}''$	5.0	91.7	....
12 15	9.17	2, 3½×2½×⅜″	14.4	264.1	97.8
14 15	9.17	2, 3½×2½×$\frac{7}{16}''$	16.6	304.4	57.7
15 15′	23.60	2, 2×2×$\frac{3}{16}''$	5.0	118.0	35.9
		Totals..................	....	10818.9	1949.8

701. Results of Preliminary Weight Sheet.—

C. L. weights......................	10818.9
Connection weights................	1949.8
Total..........................	12768.7 lbs.

And $\frac{12768.7}{16 \times 128} = 6.235$ lbs. per horizontal sq. ft. instead of 6.430 lbs. assumed by formula.

$P = 137.92\ (2 + 3 + 3 + 3 + 6.235 \cos 21.8°) = 2316$ # $= 1.158$ tons.

Therefore the P stresses are to be reduced in the proportion:

1.171 : 1.158 : : P stress : revised P stress.

But since this change only reduces the maximum stress in X 1 from 60.8 to 60.5 tons its section remains unchanged, and this is yet more the case with all other members of the truss. Therefore the preliminary weights of members and connections may be taken as the actual weights.

Then $\frac{1949.8}{10818.9} = 18\frac{2}{100}$ per cent to be added to C. L. weights for the weight of connections at apexes.

702. Comparison of Examples 5 A and 5 B.—Since the latticing of the channels should be added to the C. L. weight of the truss in Example 5 A, the C. L. weight of that truss exceeds that of Example 5 B by 11 7/10 per cent. Also the weight of its connections is greater by 11 1/2 per cent, making the total weight of 5 A exceed that of 5 B about 11 7/10 per cent. Therefore the latter type of truss is probably more economical in all cases, and especially since it requires far less skilled labor in its construction.

EXAMPLES 10 A AND B.—A STEEL SEMICIRCULAR CRESCENT TRUSS

703. General Description.—Resume Example 10 of Chapter IV, with the exception that the lower chord is subdivided into 7 equal panels, instead of unequally by radials from the centre of the upper chord. Also resume Example 10 A of Chapters XI and XII. There are two different methods of construction.

A. All members of the truss are straight, so that a splice must be made in each chord at each apex.

B. The members of the chords are curved, so that splices are only necessary at certain apexes, depending on the lengths of shapes available.

A.—CHORDS WITH STRAIGHT MEMBERS

704. Description.—Resume dimension sheet of Art. 621 of Chapter XI.

The C. L. weights are computed as before and entered on the preliminary weight sheet. Weights of the connections are computed as follows:

705. Weights of Connections.—Connection of X 1, Y 1, etc. (Figs. 627, 630).

X 1, add 0.5′ × 33.6 # = ...		16.8
2 pieces, $2\frac{1}{2} \times 2 \times \frac{3}{8}$, 1.00′ × 10.6 # = ...		10.6
2 pieces, $3\frac{1}{2} \times 3 \times \frac{3}{8}$, 1.00′ × 15.8 # = ...		15.8
2 pieces, $6 \times 6 \times \frac{3}{4}$, 1.00′ × 57.4 # = ...		57.4
Gusset, 3.40 sq. ft. × 25.5 # = ...		86.6
122 rivet heads × 1/6 # = ...		20.3
		207.5
Y 1, deduct 0.80′ × 20.8 # = ...		16.7
		190.8×2 = 381.6 #

Connection of X 1, X 2, etc. (Figs. 627, 631).

Cover, $7\frac{5}{8} \times \frac{3}{8}$, 1.50′ × 9.7 # = ...		14.6
2 pieces, $2 \times 2 \times \frac{3}{16}$, 0.4′ × 5.0 # = ...		2.0
Gusset, 0.54 sq. ft. × 25.5 # = ...		13.8
56 rivet heads × 1/6 # = ...		9.3
		39.7
Deduct 0.2′ × 5.0 # = ...		1.0
		38.7×2 = 77.4 #

Connection of Y 1, Y 3, etc. (Figs. 627, 632).

Cover, $7\frac{5}{8} \times \frac{3}{8}$, 1.3′ × 9.7 # = ...		12.7
2 pieces, $2 \times 2 \times \frac{3}{16}$, 0.4′ × 5.0 # = ...		2.0
2 pieces, $3 \times 2\frac{1}{2} \times \frac{5}{16}$, 0.7′ × 11.2 # = ...		7.9
Gusset, 1.46 sq. ft. × 25.5 # = ...		37.3
80 rivet heads × 1/6 # = ...		13.3
		73.2
Deduct 2 pieces, 0.75′ × 14.4 # = ...	10.8	
Deduct 2 pieces, 0.3′ × 5.0 # = ...	1.5	
	12.3	12.3
		61.9×2 = 123.8 #

Connection of *X* 2, *X* 4, etc. (Figs. 627, 633).

Cover, 1.5′ × 9.7 # =		14.6
2 pieces, 3 × 2½ × $\frac{5}{16}$, 0.7′ × 11.2 # =		7.9
2 pieces, 2 × 2 × $\frac{3}{16}$, 0.55′ × 5.0 # =		2.8
Gusset, 1.48 sq. ft. × 25.5 # =		37.8
76 rivet heads × 1/6 # =		12.7
		75.8
Deduct 2 pieces, 0.7′ × 14.4 # =	10.1	
Deduct 1 piece, 0.3′ × 5.0 # =	1.5	
	11.6	11.6
		64.2×2 = 128.4 #

Connection of *Y* 3, *Y* 5, etc. (Figs. 627, 634).

Cover, 1.2′ × 9.7 # =		11.7
2 pieces, 2 × 2 × $\frac{3}{16}$, 0.55′ × 5.0 # =		2.8
2 pieces, 3½ × 2½ × $\frac{5}{16}$, 0.65′ × 12.2 # =		8.0
Gusset, 1.48 sq. ft. × 25.5 # =		37.8
64 rivet heads × 1/6 # =		10.7
		71.0
Deduct 2 pieces, 0.6′ × 17.4 # =	10.5	
Deduct 2 pieces, 0.25′ × 5.0 # =	1.3	
	11.8	11.8
		59.2×2 = 118.4 #

Connection of *X* 4, *X* 6, etc. (Fig. 635).

Cover, 1.5′ × 9.7 # =		14.6
2 pieces, 3½ × 2½ × $\frac{3}{16}$, 0.6′ × 12.2 # =		7.3
2 pieces, 2½ × 2 × $\frac{5}{16}$, 0.55′ × 5.6 # =		3.1
Gusset, 1.40 sq. ft. × 25.5 # =		35.9
70 rivet heads × 1/6 # =		11.7
		72.6
Deduct 2 pieces, 0.5′ × 17.4 # =	8.7	
Deduct 2 pieces, 0.25′ × 5.6 # =	1.4	
	10.1	10.1
		62.5×2 = 125.0 #

Connection of Y 5, Y 7, etc. (Fig. 636).

Cover, 1.10′ × 9.7 # =		10.7
2 pieces, $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$, 0.6′ × 12.2 # =		7.3
2 pieces, $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{16}$, 0.55′ × 5.6 # =		3.1
Gusset, 1.54 sq. ft. × 25.5 # =		39.3
68 rivet heads × 1/6 # =		11.3
		71.7
Deduct 2 pieces, 0.25′ × 5.6 # =	1.4	
Deduct 2 pieces, 0.5′ × 17.4 # =	8.7	
	10.1	10.1
		61.6×2=123.2 #

Connection of X 6, X 8, etc. (Fig. 627).

Cover, $\frac{5}{16}$″, 1.4′ × 8.1 # =		11.4
2 pieces, $3 \times 2\frac{1}{2} \times \frac{5}{16}$, 0.6′ × 11.2 # =		6.7
2 pieces, $2\frac{1}{2} \times 2 \times \frac{1}{4}$, 0.6′ × 7.4 # =		4.4
Gusset, 1.36 sq. ft. × 25.5 # =		34.7
70 rivet heads × 1/6 # =		11.7
		68.9
Deduct 2 pieces, 0.5′ × 17.4 # =	8.7	
Deduct 2 pieces, 0.3′ × 9.8 # =	2.9	
	11.6	11.6
		57.3×2=114.6 #

Connection of Y 7, Y 9, etc. (Fig. 627).

Cover, 1.2′ × 8.1 # =		9.7
2 pieces, $2\frac{1}{2} \times 2 \times \frac{1}{4}$, 0.6′ × 7.4 # =		4.4
2 pieces, $4 \times 3 \times \frac{5}{16}$, 0.5′ × 14.4 # =		7.2
Gusset, 1.44 sq. ft. × 25.5 # =		36.8
68 rivet heads × 1/6 # =		11.3
		69.4
Deduct 2 pieces, 0.2′ × 9.8 # =	2.0	
Deduct 2 pieces, 0.3′ × 24.6 # =	7.4	
	9.4	9.4
		60.0×2=120.0 #

Connection of X 8, X 10, etc. (Fig. 627).

Cover, 1.3′ × 8.1 # =		10.5	
2 pieces, 4 × 3 × $\frac{5}{16}$, 0.4′ × 14.4 # =		5.8	
2 pieces, $2\frac{1}{2}$ × 2 × $\frac{1}{4}$, 0.5′ × 7.4 # =		3.7	
Gusset, 1.24 sq. ft. × 25.5 # =		31.6	
68 rivet heads × 1/6 # =		11.3	
		62.9	
Deduct 2 pieces, 0.45′ × 24.6 # =	11.1		
Deduct 2 pieces, 0.3′ × 9.8 # =	2.9		
	14.0	14.0	
		48.9×2 =	97.8 #

Connection of Y 9, Y 11, etc. (Fig. 627).

Cover, 1.2′ × 8.1 # =		9.7	
2 pieces, $2\frac{1}{2}$ × 2 × $\frac{1}{4}$, 0.5′ × 7.4 # =		3.7	
2 pieces, 4 × 3 × $\frac{5}{16}$, 0.55′ × 14.4 # =		7.9	
Gusset, 1.46 sq. ft. × 25.5 # =		37.3	
72 rivet heads × 1/6 # =		12.0	
		70.6	
Deduct 2 pieces, 0.2′ × 9.8 # =	2.0		
Deduct 2 pieces, 0.4′ × 24.6 # =	8.9		
	10.9	10.9	
		59.7×2 =	119.4 #

Connection of X 10, X 12, etc. (Fig. 627).

Cover, 1.4′ × 8.1 # =		11.4	
2 pieces, 4 × 3 × $\frac{5}{16}$, 0.6′ × 14.4 # =		8.7	
2 pieces, 3 × $2\frac{1}{2}$ × $\frac{1}{4}$, 0.6′ × 9.0 # =		5.4	
Gusset, 1.37 sq. ft. × 25.5 # =		35.0	
70 rivet heads × 1/6 # =		11.7	
		72.2	
Deduct 2 pieces, 0.45′ × 24.6 # =	11.1		
Deduct 2 pieces, 0.2′ × 14.4 # =	2.9		
	14.0	14.0	
		58.2×2 =	116.4 #

Connection of *Y* 11, *Y* 13, etc. (Fig. 000).

Cover, 1.3′ × 8.1 # = ...		10.6
2 pieces, 3 × 2½ × ¼, 0.55′ × 9.0 # = ...		5.0
3 pieces, 4 × 3 × $\frac{5}{16}$, 0.55′ × 14.4 # = ...		7.9
Gusset, 1.40 sq. ft. × 25.5 # = ...		35.8
68 rivet heads × 1/6 # = ...		11.3
		70.6
Deduct 2 pieces, 0.15′ × 14.4 # = ...	2.2	
Deduct 2 pieces, 0.35′ × 24.6 # = ...	8.6	
	10.8	10.8
		59.8×2 = 119.6 #

Connection of *X* 12, *X* 12′, etc. (Fig. 627).

Cover, 1.4′ × 8.1 # = ...		11.4
2 pieces, 4 × 3 × $\frac{5}{16}$, 0.6′ × 14.4 # = ...		8.7
2 pieces, 4 × 3 × $\frac{5}{16}$, 0.6′ × 14.4 # = ...		8.7
Gusset, 1.40 sq. ft. × 25.5 # = ...		35.8
68 rivet heads × 1/6 # = ...		11.3
		83.4
Deduct 2 pieces, 0.4′ × 24.6 # = ...	9.9	
Deduct 2 pieces, 0.4′ × 24.6 # = ...	9.9	
Deduct 2 pieces, 0.25′ × 5.0 # = ...	1.3	
	21.1	21.1
		62.3×1 = 62.3 #

Connection of *Y* 13, *Q* 13′, etc. (Fig. 627).

Cover, 1.3′ × 8.1 # = ...	10.6
Gusset, 0.72 sq. ft. × 25.5 # = ...	18.4
32 rivet heads × 1/6 # = ...	5.3
	34.3
Deduct 2 pieces, 0.15′ × 5.0 # = ...	0.8
	33.5×1 = 33.5 #

706. Preliminary Weight Sheet.—

Member.	C-length.	Dimensions.	Weight per foot.	C. L. Weights.	Conn. Weights.
X 1	11.20	2, 5×3½×⅝″ Ls.	33.6	752.6	+381.6
X 2	11.20	2, 5×3½×9/16″	30.4	681.0	77.4
X 4	11.20	2, 5×3½×½″	27.2	609.3	128.4
X 6	11.20	2, 5×3½×½″	27.2	609.3	125.0
X 8	11.20	2, 5×3½×7/16″	24.0	537.6	114.6
X 10	11.20	2, 5×3½×⅜″	20.8	465.9	97.8
X 12	11.20	2, 5×3½×⅜″	20.8	465.9	116.4
Y 1	9.56	2, 5×3½×⅜″	20.8	397.7	123.8
Y 3	9.56	2, 5×3½×⅜″	20.8	397.7	118.4
Y 5	9.56	2, 5×3½×5/16″	17.4	332.7	123.2
Y 7	9.56	2, 5×3½×5/16″	17.4	332.7	120.0
Y 9	9.56	2, 3½×2½×5/16″	12.2	233.3	119.4
Y 11	9.56	2, 3½×2½×5/16″	12.2	233.3	119.6
Y 13	9.56	2, 3½×2½×5/16″	12.2	233.3	33.5
1 2	3.01	2, 2×2×3/16″	5.0	30.1	....
3 4	5.94	2, 2×2×3/16″	5.0	59.4	....
5 6	7.96	2, 2½×2×3/16″	5.6	89.2	....
7 8	10.09	2, 3½×2½×¼″	9.8	197.8	....
9 10	11.42	2, 3½×2½×¼″	9.8	223.8	....
11 12	12.23	2, 4×3×5/16″	14.4	352.2	62.3
13 13′	12.50	2, 2×2×3/16″	5.0	125.0	....
2 3	12.90	2, 4×3×5/16″	14.4	371.5	....
4 5	14.36	2, 5×3½×5/16″	17.4	499.7	....
6 7	15.20	2, 5×3½×5/16″	17.4	529.0	....
8 9	16.26	2, 6×4×⅜″	24.6	800.0	....
10 11	16.51	2, 6×4×⅜″	24.6	812.3	....
12 13	16.42	2, 6×4×⅜″	24.6	807.9	....
				11180.2	1861.4

707. Results Obtained from Preliminary Weight Sheet.—

C. L. weights = ... 11180.2
Connections = ... 1861.4

Total weight = ... 13041.6 lbs.

This greatly exceeds the weight of the truss assumed by the formula, which = 7670.0 # (Art. 156). Averaging the preliminary weight of truss per sq. ft. of surface of roof, we obtain:

$$\frac{13041.6 \times 2}{100 \times 16 \times \pi} = 5.189 \text{ \# per sq. ft of roof.}$$

$P = 179.52(2+4+0+4+5.189) = 2727$ # $= 1.364$ tons instead of 1.172.

Therefore the P stresses are to be increased in the proportion of
1.172 : 1.364 : : P stress : revised P stress.

The P stresses are then computed, as well as the corresponding − and + maximum stresses and entered on the Revised Weight Sheet. It then becomes necessary to determine anew the safe dimensions of the members in the same manner as before, entering these and their weights per ft. on the same sheet. Their C. L. weights are next computed and entered. The details of connections and of their weight will probably not be changed.

708. Revised Weight Sheet.—

Member.	C-length.	Dimensions.	Weight per foot.	C. L. Weights.	Connections.
X 1	11.20	2, $5\times3\frac{1}{2}\times\frac{11}{16}''$ Ls.	36.6	819.8	+381.6
X 2	11.20	2, $5\times3\frac{1}{2}\times\frac{5}{8}''$	33.6	752.6	77.4
X 4	11.20	2, $5\times3\frac{1}{2}\times\frac{1}{2}''$	27.2	609.3	128.4
X 6	11.20	2, $5\times3\frac{1}{2}\times\frac{1}{2}''$	27.2	609.3	125.0
X 8	11.20	2, $5\times3\frac{1}{2}\times\frac{7}{16}''$	24.0	537.6	114.6
X 10	11.20	2, $5\times3\frac{1}{2}\times\frac{3}{8}''$	20.8	465.9	97.8
X 12	11.20	2, $5\times3\frac{1}{2}\times\frac{3}{8}''$	20.8	465.9	116.4
Y 1	9.56	2, $5\times3\frac{1}{2}\times\frac{3}{8}''$	20.8	397.7	123.8
Y 3	9.56	2, $5\times3\frac{1}{2}\times\frac{3}{8}''$	20.8	397.7	118.4
Y 5	9.56	2, $5\times3\frac{1}{2}\times\frac{5}{16}''$	17.4	332.7	120.0
Y 7	9.56	2, $5\times3\frac{1}{2}\times\frac{5}{16}''$	17.4	332.7	120.0
Y 9	9.56	2, $3\frac{1}{2}\times2\frac{1}{2}\times\frac{3}{8}''$	14.4	275.3	119.4
Y 11	9.56	2, $3\frac{1}{2}\times2\frac{1}{2}\times\frac{3}{8}''$	14.4	275.3	119.6
Y 13	9.56	2, $3\frac{1}{2}\times2\frac{1}{2}\times\frac{3}{8}''$	14.4	275.3	33.5
1 2	3.01	2, $2\times2\times\frac{3}{16}''$	5.0	30.1	....
3 4	5.94	2, $2\times2\times\frac{3}{16}''$	5.0	59.4	....
5 6	7.96	2, $2\times2\times\frac{1}{4}''$	6.4	101.9	....
7 8	10.09	2, $2\times2\times\frac{1}{4}''$	6.4	129.2	....
9 10	11.42	2, $3\frac{1}{2}\times2\frac{1}{2}\times\frac{1}{4}''$	9.8	223.8	....
11 12	12.23	2, $4\times3\times\frac{5}{16}''$	14.4	352.2	62.3
13 13′	12.50	2, $2\times2\times\frac{3}{16}''$	5.0	125.0	....
2 3	12.90	2, $4\times3\times\frac{5}{16}''$	14.4	371.5	....
4 5	14.36	2, $5\times3\frac{1}{2}\times\frac{5}{16}''$	17.4	499.7	....
6 7	15.20	2, $5\times3\frac{1}{2}\times\frac{5}{16}''$	17.4	529.0	....
8 9	16.26	2, $6\times4\times\frac{3}{8}''$	24.6	800.0	....
10 11	16.51	2, $6\times4\times\frac{3}{8}''$	24.6	812.3	....
12 13	16.42	2, $6\times4\times\frac{3}{8}''$	24.6	807.9	....
				11504.4	1852.7

709. Results of Revised Weight Sheet.—

C. L. weights = ... 11504.4
Connections = ... 1852.7

Total weight = ... 13357.1 lbs.

This makes an increase in the weight of the truss of but 324.2 #, so that it is evidently unnecessary to make a second revision of weights, and the revised weight may be taken as the actual weight of the truss.

Then $\frac{1852.7}{11504.4} = 16\frac{10}{100}$ per cent to be added to C. L. weights for connections.

Also $\frac{13357.1 - 7670}{7670} = 74\frac{14}{100}$ per cent excess of weight of truss over weight assumed by formula. This excess is due to several causes.

1. Arched crescent form of the truss.
2. Greatly increased area of roof surface.
3. Much greater total wind pressure on the roof.

This type of truss is evidently much more expensive than any triangular type of truss for an equal span.

B.—CHORDS WITH CURVED MEMBERS

710. Description.—This truss is similar to 10 A in all respects, excepting that the chords are curved to the required circular arcs between the apexes instead of being straight. Resume dimension sheet of Art. 621, Chapter XI. It is evident that splices in the chords need not occur at each apex, and that the cross-section must remain constant between splices. The ends of the wooden purlins do not require boxing, since each one rests directly on the flanges of the upper chord.

711. Weights of Connections at Apexes.—Resume details of connections given in Chapter XII.

Connection of X 1, Y 1 (Fig. 637).

2 pieces, $5 \times 3\frac{1}{2} \times \frac{7}{16}$, $0.5' \times 30.4$ # = ... 15.2
2 pieces, $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{8}$, $0.6' \times 14.4$ # = ... 8.7
2 pieces, $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{8}$, $0.8' \times 14.4$ # = ... 11.5
Gusset, 2.90 sq. ft. $\times$ 25.5 # = ... 74.0
86 rivet heads $\times$ 1/6 # = ... 14.7

124.1
Deduct 2 pieces, $5 \times 3\frac{1}{2} \times \frac{7}{16}$, $0.8' \times 24.0$ # = ... 19.5

$104.6 \times 2 = 209.2$ #

Connection of X 1, X 2, etc. (Fig. 637).

Gusset, 0.20 sq. ft. × 25.5 # = ...		5.0	
8 rivet heads × 1/6 # = ...		1.3	
		6.3	
Deduct 0.30′ × 5.0 # = ...		1.5	
		4.8×2=	9.6 #

Connection of Y 1, Y 3, etc. (Fig. 637).

2 pieces, 3 × 2½ × ¼, 0.7′ × 9.0 # = ...		6.3	
Gusset, 1.20 sq. ft. × 25.5 # = ...		30.6	
42 rivet heads × 1/6 # = ...		7.0	
		43.9	
Deduct 0.25′ × 5.0 # = ...	1.3		
Deduct 0.90′ × 14.4 # = ...	13.0		
	14.3	14.3	
		29.6×2=	59.2 #

Connection of X 2, X 4, etc. (Fig. 637).

2 pieces, 3 × 2½ × ¼, 0.6′ × 9.0 # = ...		5.4	
Gusset, 0.80 sq. ft. × 25.5 # = ...		20.4	
24 rivet heads × 1/6 # = ...		4.0	
		29.8	
Deduct 0.7′ × 14.4 # = ...	10.1		
Deduct 0.3′ × 5.0 # = ...	1.5		
	11.6	11.6	
		16.2 #	

Connection of Y 3, Y 5, etc. (Fig. 637).

2 pieces, 3½ × 2½ × ¼, 0.5′ × 9.8 # = ...		4.9	
Gusset, 1.0 sq. ft. × 25.5 # = ...		25.5	
40 rivet heads × 1/6 # = ...		6.7	
		37.1	
Deduct 0.3′ × 5.0 # = ...	1.5		
Deduct 0.55′ × 17.4	9.6		
	11.1	11.1	
		26.0×2=	52.0 #

Connection of X 4, X 6, etc. Splice (Fig. 637).

2 pieces, $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$, $0.7' \times 9.8$ # = ...		6.2
2 pieces, $2 \times 2 \times \frac{3}{16}$, $0.7' \times 5.0$ # = ...		3.5
Cover, $7\frac{5}{8} \times \frac{1}{2}$, $1.10' \times 13.0$ # = ...		14.5
Gusset, 1.20 sq. ft. $\times$ 25.5 # = ...		30.6
70 rivet heads $\times$ 1/6 # = ...		11.7
		66.5
Deduct $0.5' \times 17.4$ # = ...	8.7	
Deduct $0.25' \times 5.6$ # = ...	1.4	
	10.1	10.1
		$56.4 \times 2 = 112.8$ #

Connection of Y 5, Y 7, etc. Splice (Fig. 637).

2 pieces, $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$, $0.65' \times 9.8$ # = ...		6.4
2 pieces, $2 \times 2 \times \frac{3}{16}$, $0.6' \times 5.0$ # = ...		3.0
Cover, $7\frac{5}{8} \times \frac{1}{2}$, $1.10' \times 13.0$ # = ...		14.3
Gusset, 1.40 sq. ft. $\times$ 25.5 # = ...		35.8
72 rivet heads $\times$ 1/6 # = ...		12.0
		71.5
Deduct $0.25' \times 5.6$ # = ...	1.4	
Deduct $0.55' \times 17.4$ # = ...	9.6	
	11.0	11.0
		$60.5 \times 2 = 121.0$ #

Connection of X 6, X 8, etc. (Fig. 637).

2 pieces, $3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$, $0.6' \times 9.8$ # = ...		3.9
2 pieces, $2\frac{1}{2} \times 2 \times \frac{1}{4}$, $0.5' \times 7.4$ # = ...		3.7
Gusset, 0.96 sq. ft. $\times$ 25.5 # = ...		24.5
52 rivet heads $\times$ 1/6 # = ...		8.7
		40.8
Deduct $0.4' \times 17.4$ # = ...	7.0	
Deduct $0.25' \times 9.8$ # = ...	2.5	
	9.5	9.5
		$31.3 \times 2 = 62.6$ #

Connection of Y 7, Y 9, etc. (Fig. 637).

2 pieces, $2\frac{1}{2} \times 2 \times \frac{1}{4}$, 0.6′ × 7.4 # = ...		4.4	
2 pieces, $4 \times 3 \times \frac{5}{16}$, 0.6′ × 14.4 # = ...		8.6	
Gusset, 1.34 sq. ft. × 25.5 # = ...		34.2	
58 rivet heads × 1/6 # = ...		9.7	
		56.9	
Deduct 0.3′ × 9.8 # = ...	2.9		
Deduct 0.6′ × 24.6 # = ...	14.8		
	17.7	17.7	
		39.2×2=	78.4 #

Connection of X 8, X 10, etc. (Fig. 637).

2 pieces, $4 \times 3 \times \frac{5}{16}$, 0.7′ × 14.4 # = ...		10.1	
2 pieces, $2\frac{1}{2} \times 2 \times \frac{1}{4}$, 0.5′ × 7.4=		3.7	
Gusset, 1.14 sq. ft. × 25.5 # = ...		29.3	
56 rivet heads × 1/6 # = ...		9.3	
		52.4	
Deduct 0.4′ × 24.6 # = ...	9.9		
Deduct 0.25′ × 9.8 # = ...	2.5		
	12.4	12.4	
		40.0×2=	80.0 #

Connection of Y 9, Y 11, etc. (Fig. 637).

2 pieces, $2\frac{1}{2} \times 2 \times \frac{1}{4}$, 0.6′ × 7.4 # = ...		4.4	
2 pieces, $4 \times 3 \times \frac{5}{16}$, 0.6′ × 14.4 # = ...		8.6	
Gusset, 1.20 sq. ft. × 25.5 # = ...		30.6	
54 rivet heads × 1/6 # = ...		9.0	
		52.6	
Deduct 0.25′ × 9.8 # = ...	2.5		
Deduct 0.45′ × 24.6 # = ...	11.1		
	13.6	13.6	
		39.0×2=	78.0 #

Connection of X 10, X 12, etc. Splice (Fig. 637).

2 pieces, $4 \times 3 \times \frac{5}{16}$, $0.65' \times 14.4$ # =		9.4
2 pieces, $3 \times 2\frac{1}{2} \times \frac{1}{4}$, $0.5' \times 9.0$ # =		4.5
Cover, $7\frac{5}{8} \times \frac{1}{2}$, $1.10' \times 13.0$ # =		14.5
Gusset, 1.40 sq. ft. $\times 25.5$ # =		35.8
80 rivet heads $\times 1/6$ # =		13.3
		77.5
Deduct $0.55' \times 24.6$ # =	12.3	
Deduct $0.25' \times 14.4$ # =	3.6	
	15.9	15.9
		$61.6 \times 2 = 123.2$ #

Connection of Y 11, Y 13, etc. Splice (Fig. 637).

2 pieces, $3 \times 2\frac{1}{2} \times \frac{1}{4}$, $0.6' \times 9.0$ # =		5.4
2 pieces, $4 \times 3 \times \frac{5}{16}$, $0.7' \times 14.4$ # =		10.1
Cover, $1.10' \times 13.0$ # =		14.5
Gusset, 1.54 sq. ft. $\times 25.5$ # =		39.3
72 rivet heads $\times 1/6$ # =		12.0
		81.3
Deduct $0.3' \times 14.4$ # =	4.3	
Deduct $0.4' \times 24.6$ # =	9.8	
	14.1	14.1
		$67.2 \times 2 = 134.4$ #

Connection of X 12, X 12′, etc. (Fig. 637).

2 pieces, $4 \times 3 \times \frac{5}{16}$, $0.7' \times 14.4$ # =		10.1
2 pieces, $4 \times 3 \times \frac{5}{16}$, $0.7' \times 14.4$ # =		10.1
Gusset, 1.56 sq. ft. $\times 25.5$ # =		39.8
68 rivet heads $\times 1/6$ # =		10.3
		70.3
Deduct $0.4' \times 24.6$ # =	9.9	
Deduct $0.4' \times 24.6$ # =	9.9	
Deduct $0.8' \times 5.0$ # =	4.0	
	23.8	23.8
		$46.5 \times 1 = 46.5$ #

Connection of Y 13, Y 13′, etc. (Fig. 637).

Gusset, 0.51 sq. ft. × 25.5 # = ...	13.0	
10 rivet heads × 1/6 # = ...	1.7	
	14.7	
Deduct 0.15′ × 5.0 # = ...	0.7	
	14.0×1 =	14.0 #

712. Preliminary Weight Sheet.—

Member.	C-length.	Dimensions.	Weight per foot.	C. L. Weights.	Connections.
X 1	11.25	2, 5×3½×$\frac{9}{16}$″ Ls.	30.4	684.0	+209.2
X 2	11.25	2, 5×3½×$\frac{9}{16}$″	30.4	684.0	9.6
X 4	11.25	2, 5×3½×$\frac{9}{16}$″	30.4	684.0	36.4
X 6	11.25	2, 5×3½×$\frac{7}{16}$″	24.0	540.0	112.8
X 8	11.25	2, 5×3½×$\frac{7}{16}$″	24.0	540.0	62.6
X 10	11.25	2, 5×3½×$\frac{7}{16}$″	24.0	540.0	80.0
X 12	11.25	2, 5×3½×⅜″	20.8	468.0	77.5
Y 1	9.58	2, 5×3½×$\frac{7}{16}$″	24.0	459.8	59.2
Y 3	9.58	2, 5×3½×$\frac{7}{16}$″	24.0	459.8	52.0
Y 5	9.58	2, 5×3½×$\frac{7}{16}$″	24.0	459.8	121.0
Y 7	9.58	2, 5×3½×$\frac{5}{16}$″	17.4	333.4	78.4
Y 9	9.58	2, 5×3½×$\frac{5}{16}$″	17.4	333.4	78.0
Y 11	9.58	2, 5×3½×$\frac{5}{16}$″	17.4	333.4	134.0
Y 13	9.58	2, 5×3½×$\frac{5}{16}$″	17.4	333.4	14.0
1 2	3.01	2, 2×2×$\frac{3}{16}$″	5.0	30.1	....
3 4	5.94	2, 2×2×$\frac{3}{16}$″	5.0	59.4	....
5 6	7.96	2, 2½×2×$\frac{3}{16}$″	5.6	89.2	....
7 8	10.09	2, 3½×2½×¼″	9.8	197.8	....
9 10	11 42	2, 3½×2½×¼″	9.8	223.8	....
11 12	12.23	2, 4×3×$\frac{5}{16}$″	14.4	352.2	....
13 13′	12.50	2, 2×2×$\frac{3}{16}$″	5.0	62.5	70.3
2 3	12.90	2, 4×3×$\frac{5}{16}$″	14.4	371.5	....
4 5	14.36	2, 5×3½×$\frac{5}{16}$″	17.4	499.7	....
6 7	15.20	2, 5×3½×$\frac{5}{16}$″	17.4	529.0	....
8 9	16.26	2, 6×4×⅜″	24.6	800.0	....
10 11	16.51	2, 6×4×⅜″	24.6	812.3	....
12 13	16.42	2, 6×4×⅜″	24.6	807.9	....
				11688.4	1195.0

713. Results of Preliminary Weight Sheet.—

C. L. weights = ...	11688.4
Connections = ...	1195.0
Total weight = ...	12882.4 lbs.

Then $\dfrac{12882.4 \times 2}{100 \times 16 \times \pi} = 5.126$ lbs. per sq. ft. of surface of roof.

$P = 179.52\,(2 + 4 + 0 + 4 + 5.126) = 2715$ lbs. $= 1.308$ tons. The P stresses are then to be increased in the proportion of: $1.308 : 1.172 :: P$ stresses : revised P stresses.

The dimensions and C. L. weights are then revised as for the last example, and this adds 205 # to the C. L. weights, since but few members require larger sections. This produces a total weight of 13088.4 lbs., that may be taken as the actual weight of the truss.

Then $\dfrac{1195.0}{11893.4} = 10\dfrac{5}{100}$ per cent to be added to C. L. weight for the weight of connections.

Also $\dfrac{13089.4 - 7670}{7670} = 70\dfrac{65}{100}$ per cent excess of total weight of truss over the weight assumed by the formula. This results from the causes mentioned for Example 10 A.

714. Comparison of Examples 10 A and 10 B.—This truss is slightly lighter than the one with straight chord members, owing to the simpler construction at apexes and the reduced weight of the connections.

From Examples 10 A and B, it is evident that the semicircular crescent truss is much more expensive than the ordinary triangular type, although its internal appearance may be desirable for train-sheds, armories, etc.

CHAPTER XV

EXAMPLE 26. A COMPLETE STUDY OF A MODIFIED FINK TRUSS

715. Description.—This truss is taken as an example of the complete computations of loads, determination of maximum stresses in members, dimensioning members, detailing connections at apexes, and computation and revision of weights of members and connections, just as it is most conveniently practised in ordinary cases.

716. Programme.—Truss of the type shown in Fig. 639: span 150 ft.; rise of upper chord, 25 ft.; materials, steel excepting 7/8″ longleaf pine sheathing and a strip 1 3/4″ square fastened on top of each steel rafter to receive the wooden sheathing; covering of felt, asphalt, and gravel; trusses 20 ft. on centres; location at St. Louis, Mo., latitude about 38.6′ north; medium exposure.

717. Dimensions.—

$$\text{Tan. } i^\circ = \frac{25}{75} = 0.3333 = \tan 18^\circ\ 26'\ 6'' = 18.4^\circ \text{ approximately.}$$

Length of principal $= \sqrt{75^2 + 25^2} = 79'\ 0\ 11/16''$.

Half length of principal $= 39'\ 6\ 11/32''$.

Sixth length of principal $= 13'\ 2\ 1/8''$.

Member 5 6 $= (39'\ 6\ 11/32'') \times \tan 18^\circ\ 26'\ 6'' = 13'\ 2\ 1/4''$.

Member 1 2 = 9 10 = one-third of 5 6 $= 4'\ 4\ 3/4''$.

Member 3 4 = 7 8 = two-thirds of 5 6 $= 8'\ 9\ 1/2''$.

Member Y 1 = Y 3 = Y 5 = 6 11 = 8 11 = 10 11 =

$$\sqrt{(13'\ 2\ 1/4'')^2 + (4'\ 4\ 3/4'')^2} = 13'\ 10\ 13/16''.$$

Member Y 11 $= 75'\ 0'' - 3\ (13'\ 10\ 13/16'') = 33'\ 3\ 9/16''$.

Member 2 3 = 8 9 = Y 1 $= 13'\ 10\ 13/16''$.

Member 4 5 = 6 7 $= \sqrt{(13'\ 2\ 1/4'')^2 + (8'\ 9\ 1/2'')^2} = 15'\ 10\ 3/16''$.

Member 11 11′ $= 25'\ 0''$.

l' = one-sixth principal $= 13'\ 2\ 1/8''$.

$A = 263.52$ sq. ft.

718. Truss, Snow, and Wind.—

Truss $= \frac{150}{25} + \frac{150^2}{12600} = 7.786$ lbs. per horizontal sq. ft.

Snow $= 2.5\,(38.6° - 35.00) = 9\,.00$ lbs. per horizontal sq. ft.

Wind $= 8\ 9 \times 18.4° = 16.36$ lbs. per sq. ft. of roof surface.

719. Sheathing.—

Felt, asphalt, and gravel covering	6 #
Sheathing 7/8″ thick	4 #
	10 #

Then $w' = 10 \cos 18.4° + 16.36 = 26.13$ # per sq. ft. of roof, normal to it.

The parallel component is resisted by the sheathing edgewise and may be neglected.

Apply formulas 107 and 110, Art. 454.

$$L = 51.6\,t\sqrt{\frac{F}{w'}} = 51.6 \times 0.875\sqrt{\frac{0.70}{26.13}} = 7.37 \text{ ft. O. C. rafters.}$$

$$L = 1.44\,t\sqrt[3]{\frac{E}{w'}} = 1.44 \times 0.875\sqrt[3]{\frac{850}{26.13}} = 4.01 \text{ ft. O. C. rafters.}$$

Hence the rafters cannot be spaced over 4 ft. on centres.

720. Rafters.—Steel with 1 3/4″ sq. wood strips on top. Try rafters 4 ft. on centres and determine dimensions.

$w' = \cos 18.4°\,(6 + 4 + 4) + 16.36 = 29.6$ # = normal component.

$w'' = \sin 18.4°\,(6 + 4 + 4) = 4.42$ # = parallel component.

Apply formulas 97 and 101, Art. 453.

$\frac{I}{c} = \frac{w' L^2 e}{128000} = \frac{29.6 \times 13.18^2 \times 48''}{128000} = 1.94$, corresponds to 1, 4″ 6 # channel.

$I = \frac{w' L^3 e}{515556} = \frac{29.6 \times 13.18^3 \times 48''}{515556} = 6.35$, corresponds to 1, 5″ 6 1/2 # channel.

This section probably has sufficient surplus strength to safely resist the longitudinal compression in the rafter produced by the parallel component w''.

Check this by formulas 140 and 141, Art. 470.

$W'' = \frac{4.42 \times 13.18 \times 4}{2000} = 0.118$ ton. A' = sectional area of rafter.

Then $\frac{W''}{2A'} = \frac{0.118}{2 \times 1.95} = 0.03$ ton per sq. in. of section of rafter.

Applying formula 140, Art. 470:

$$W' = \frac{29.6 \times 13.18 \times 4.00}{2000} = 0.723 \text{ ton} = \text{normal load on rafter.}$$

$\Delta = \frac{22.5\, W' L^3}{E\, I} = \frac{22.5 \times 0.723 \times 13.18^3}{14500 \times 7.4} = 0.347'' =$ deflection of rafter.

Apply formula 141, Art. 470:

$\frac{W''}{2A'}\left(1 + \frac{6\,\Delta}{d}\right) = 0.03\left(1 + \frac{6 \times 0.347}{5}\right) = 0.43$ ton per sq. inch maximum fibre stress in section at middle of length of rafter.

Hence this channel is amply safe for the rafter.

Since the sheathing of the roof might sag permanently between the rafters sufficiently to look badly, it would probably be preferable to space the rafters more closely. Try them 2 1/2 ft. on centres, making 8 spaces per bay of the roof instead of but 5.

Applying formulas 97 and 101, Art. 453:

$\frac{I}{c} = \frac{w' L^2 e}{128000} = \frac{29.6 \times 13.18^2 \times 30''}{128000} = 1.21$, corresponding to 1, 4″ 5 1/4 # channel.

$I = \frac{w' L^3 e}{51556} = \frac{29.6 \times 13.18^3 \times 30''}{51556} = 4.98$, corresponding to 1, 5″ 6 1/2 # channel.

Hence the same channel section is required as if the rafters were spaced 4 ft. on centres. But, although requiring a greater weight of steel, this arrangement will be preferable to the former, since the sheathing cannot sag between the rafters. A channel section is better than an *I*-section, because there is more space for rivets connecting the wooden strip to the rafter and fastening the rafter to the purlin, and further, since the channel section is considerably lighter for equal strength and stiffness.

Then $\frac{6.5}{2.5} = 2.60$ # per sq. ft. = actual weight of rafters.

721. Purlins.—
$w' = \cos 18.4° \,(6+4+2.6+3) + 16.36\# = 31.16$ = normal component.
$w'' = \sin 18.4°(6+4+2.6+3) = 4.92$ = parallel component.
$W' = 31.16 \times 263.52 = 8211\# = 4.106$ tons = normal load on purlin.
$W'' = 4.92 \times 263.52 = 1298\# = 0.649$ ton = parallel load on purlin.

Apply formulas 83 and 86, Art. 450, for both W' and W'':

$\frac{I}{c} = \frac{3W'L}{16} = \frac{3 \times 4.106 \times 20}{16} = 15.48$, corresponding to 2, 8″ 11 1/2 # channels.

$$\frac{I}{c} = \frac{3W''L}{16} = \frac{3 \times 0.649 \times 20}{16} = 2.46.$$

$I = 0.0466\, W' L^2 = 0.0466 \times 4.106 \times 20^2 = 76.9$, corresponding to 2, 9″ 13 1/4 # channels.

$I = 0.0466\, W'' L^2 = 0.0466 \times 0.649 \times 20^2 = 12.2$.

Hence each purlin must be composed of 2, 9″ 13 1/4 # channels, which must be spaced about 6.54 ins. apart and latticed together on top and bottom flanges. This makes them equally stiff, both normal and parallel to the roof surface. Then $2 \times 2.43 + 5.64 = 10.5''$ = extreme width of the purlin across the flanges.

Check maximum fibre stress in purlin by applying formula 145, Art. 476. Here $I_x = 2\,[1.77 + 3.89\,(2.82 + 0.61)^2] = 95.14$.

$.75\, L \left(\frac{W'd}{I_y} + \frac{W''b}{I_x}\right) = 0.75 \times 20 \left(\frac{4.106 \times 9}{2 \times 47.3} + \frac{0.0649 + 10.5}{95.14}\right) =$ 5.99 tons per sq. inch, which is more than safely below the limiting safe value of 8.00 tons per sq. inch.

These channels might safely be set closer, but this would be less convenient for latticing them.

Then $\frac{2 \times 11\,1/4 \times 20}{263.52} = 1.708\#$ per sq. ft. of roof instead of 3 #, but exclusive of the weight of the latticing. We will assume 2 # per sq. ft. to include latticing.

722. Apex Loads.—
$P = 263.52\,(6 + 4 + 2.6 + 2 + 7.786 \cos 18.4°) = 5792$ lbs. = 2.896 tons.
$S = 263.52 \times 9 \cos 18.4° = 2248$ lbs. = 1.124 tons.
$W = 263.52 \times 16.36\# = 4311$ lbs. = 2.156 tons.

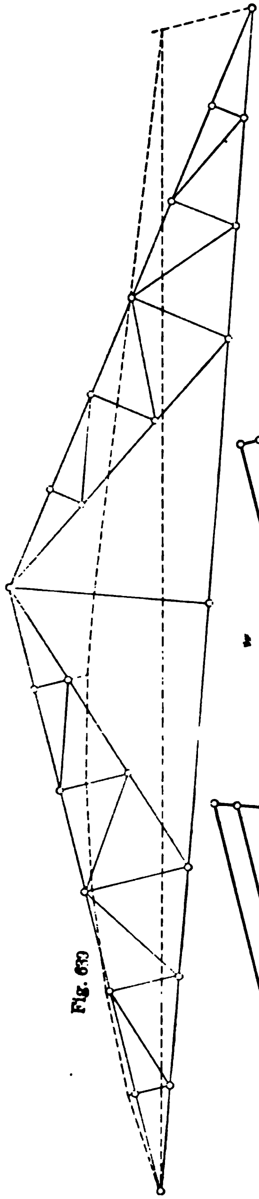


Fig. 639

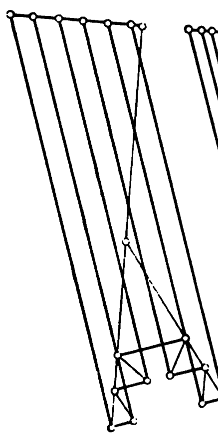


Fig. 640

Fig. 641

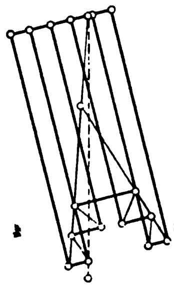


Fig. 642

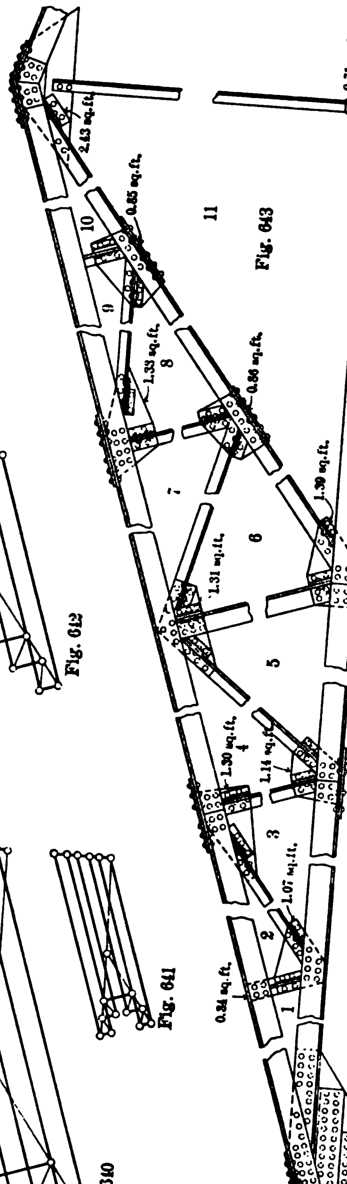


Fig. 643

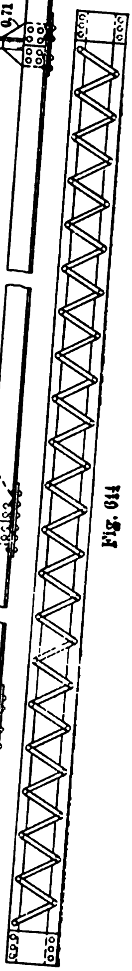


Fig. 644

723. Loads on Half Truss.—

Permanent = 2.896 × 5 1/2 = 15.93 tons.
Snow = 1.124 × 5 1/2 = 6.18 tons.
Wind = 2.156 × 5 1/2 = 11.59 tons.

724. Stress Diagrams.—Fig. 639 represents the truss diagram; Fig. 640 is the *P* stress, Fig. 641 the *S* stress, and Fig. 642 the *W* stress diagrams. These are drawn in the usual manner, the stresses are measured and entered on the Preliminary Stress, Dimension, and Weight Sheet. The maximum stress here consists of the sum of *P* and *W* stresses, the *S* stress being much less than the *W* stress.

725. Dimensioning the Members.—Since the truss is entirely constructed of pairs of steel angles, Tables M, N, and O are used for members in compression, Table G for those in tension, and Table T for determining the number of 3/4 in. rivets required at each end of each member. The use of these tables has been fully explained in Chapter X. Splices are made as indicated in Fig. 643.

Fig. 644 represents the top or bottom of one of the latticed purlins with batten plates connecting the flanges at each end of purlin.

726. Detailing Connections at Apexes.—These are made as described in Chapter XII, using 3/4 in. rivets, 5/8 in. gussets, and covers at least 5/16″ thick at splices, excepting where a greater thickness is required for transmitting the stress in the flanges of the member. Both legs of each angle are assumed to be riveted, excepting for members 1 2 and 9 10, where connection of one leg suffices.

727. Computing C. L. Weights of Members.—This is to be done as in Chapter XIV, then entering the weights on the Preliminary Weight Sheet.

728. Computing Weights of Connections at Apexes.—These are separately computed for each apex as in Chapter XIV, then entered in the proper column of the Preliminary Weight Sheet.

Expansion rolls would be required at one end of the truss, as well as cast-iron seats at each end, but their weights are not included in this computation. Their arrangement would be as already described in a previous chapter.

729. Preliminary Stress, Dimension, and Weight Sheet.—

Member.	P-stress.	S-stress.	W-stress.	Maximum.	C-length.	Dimensions.	Weight per foot.	C. L. Weight.	Connection.
X 1	−50.4	−19.6	−25.2	−75.6	13.18′	2, 6×6×$\frac{11}{16}$″Ls	53.0	1397.1	1040.6
X 2	−49.6	−19.2	−25.2	−74.8	13.18	2, 6×6×$\frac{11}{16}$″	53.0	1397.1	17.0
X 4	−44.4	−17.2	−21.9	−66.3	13.18	2, 6×6×$\frac{9}{16}$″	43.8	1154.6	139.2
X 7	−43.6	−16.9	−21.9	−65.5	13.18	2, 6×6×$\frac{9}{16}$″	43.8	1154.6	104.4
X 9	−46.8	−18.2	−25.2	−72.0	13.18	2, 6×6×⅝″	48.4	1275.8	140.6
X 10	−45.9	−17.8	−25.2	−71.1	13.18	2, 6×6×⅝″	48.4	1275.8	17.0
Y 1	+47.8	+18.6	+27.7	+75.5	13.90	2, 6×6×⅝″	48.4	1345.5	76.8
Y 3	+43.3	+16.8	+24.2	+67.5	13.90	2, 6×6×⅝″	48.4	1345.5	108.4
Y 5	+39.1	+15.2	+20.9	+60.0	13.90	2, 6×6×½″	39.2	1089.8	162.8
Y 11	+26.2	+10.2	+10.5	+36.7	33.30	2, 6×3½×⅜″	23.4	1558.4	201.5
1 2	− 2.8	− 1.1	− 2.2	− 5.0	4.40	2, 2×2×$\frac{3}{16}$″	5.0	44.0	.....
3 4	− 4.1	− 1.6	− 3.2	− 7.3	8.79	2, 3×3×$\frac{5}{16}$″	12.2	214.5	.....
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7 8	− 4.1	− 1.6	− 3.2	− 7.3	8.79	2, 3×3×$\frac{5}{16}$″	12.2	214.5	.....
9 10	− 2.8	− 1.1	− 2.2	− 5.0	4.40	2, 2×2×$\frac{3}{16}$″	5.0	44.0	.....
11 11′	+ 0.0	+ 0.0	+ 0.0	+ 0.0	25.00	2, 2×2×$\frac{3}{16}$″	5.0	125.0	124.4
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10 11	+21.6	+ 8.4	+17.1	+38.7	13.90	2, 4×4×$\frac{7}{16}$″	2.6	628.3	.....
								16467.3	2355.5

730. Results of Preliminary Stress, Dimension, and Weight Sheet.—

C. L. weights = ... 16467.3
Connections = ... 2355.5

Total weight of truss = ... 18822.8 lbs.

Then $\frac{18822.8}{150 \times 20} = 6.274$ # per horizontal sq. ft. instead of 7.785 # assumed by formula for weight of truss.

$P = 263.52(6+4+2.6+2.0+6.274 \cos 18.4°) = 5416$ lbs. $= 2.708$ tons.

Then 2.896 : 2.708 : : P stress : revised P stress.

These revised P stresses are then entered on the revised stress, dimension, and weight sheet, the S and W stresses being unchanged. Maximum stress $= P + W$ stresses. The dimensions are then

determined again, and it is found that only those of X 1, X 2, and 8 11 can be reduced with a saving of 331.6 lbs. The weights of the connections would be unchanged.

731. Results of Revised Stress, Dimension, and Weight Sheet.—

Hence preliminary C. L. weights = ..	16467.3
Reduction of C. L. weights = ...	331.6
Revised C. L. weights = ...	16135.7
Connections as before = ...	2355.5
Revised total weight of truss	18491.2 lbs.

It is evidently unnecessary to repeat the process of revision.

Then $\frac{2355.7}{16135.7}$ = 14 3/5 per cent to be added to the centre length weights for the weights of the connections at the apexes of the truss.

INDEX

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